DESIGN

of

A Double Leaf Trunnioned Deck Bascule Highway Bridge with Cantilever Type Deck Plate Girder Approach Spans.

A THESIS

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MASTER OF SCIENCE

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By

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FOREWORD

Most bascule bridges are ugly structures, and with this fact in mind, the writer has endeavored, in this design, to produce a bascule type bridge, having not only stability and economy, but something of grace and beauty also.

The bascule span in this project embodies structural features that the writer has observed on various bascule bridges now in operation. Much difficulty was experienced in obtaining methods of design, since most authors of bridge engineering text-books, after having described and compared various types of bascule and draw bridges, pass over the question of design entirely. Therefore, many of the design problems had to be worked out without recourse to text-books, and, therefore, will perhaps not appear conventional to one who is familiar with the practice of bascule bridge design.

The approach spans are rather unusual in design, and afforded a very interesting problem. Being of the cantilever type, a statically determinate structure was possible, at the same time, making possible the multiple span arch outline which was desired for this structure.

THE BASCULE BRIDGE

EARLY TYPES:

The modern bascule span is a descendent of the medieval drawbridge. It then served a double purpose, bridging the moat whe lowered, and barricading the castle doorway when raised. This earl drawbridge was not counterweighted to any appreciable extent. It was raised by hand by means of an out-haul line attached to the free end of the span, and passing upward and over a system of pulle This early bridge could not be called a bascule, since the word "Bascule" comes from the french, meaning a balance.

Due to the lack of convenient power for operating and the crude and inefficient methods of counterweighting, very little progress was made in bascule bridge development until comparatively recent years. The real beginning of the modern bascule began in th 19th century, and really can be said to have begun not over forty years ago. The Tower Bridge of London, constructed during the year 1894 probably marked the beginning.

INHERENT ADVANTAGES IN THE BASCULE BRIDGE:

Rapidity and extent of opening to permit the passage of vessels and closing again for bridge traffic, has been a great fact in the success and popularity of the modern bascule bridge. For sm vessels, the bascule may be raised slightly, thereby interrupting bridge traffic for a proportionally less time than that required fo full opening. Too, river traffic can approach to within a very sho distance of the bridge while waiting for it to open. Where river traffic is congested, the time saving thus effected is valuable in reducing delay, both to river and bridge traffic.

MODERN TYPES:

Modern bascules are comprised in three classes -

The trunnion type; The rolling lift type, and The roller bearing type. Any of these may be single or double leaf; through spans or deck spans trussed or girders. In the trunnioned type, the trunnions are placed near the center of gravity of the entire moving leaf. In the rolling lift type, the center of rotation changes continually throughout the entire period of opening and closing. The center of gravity of the moving leaf travelling backward and forward on a horizontal line. Th: changes the position of the reaction of the superstructure on the pie: throughout the period of opening and closing of the bridge.

In the roller bearing type, the trunnion is eliminated. The center of rotation is coincident with the center of gravity of the moving leaf. The load is carried by means of roller bearings on a curved track. This device allows the stress to be distributed over a greater area, thereby reducing the unit bearing stress, and decreas ing the frictional resistance due to rotation.

There are four principle types of bascule bridges in general use at the present time:- The Hall; The Chicago or Simple Trunnion type; The Scherzer; and The Strauss. The Scherzor type lead in the number in operation, the Strauss next; The Hall having the few est in operation.

DESIGN AND ERECTION PROBLEMS:

As the name implies, the bascule bridge is a balanced moving span. In order that the operating machinery may be as light as possible, it is essential that the moving leaf be in almost perfec balance. The only loads then on the operating machinery will be, those due to inertia, wind and friction. It is necessary, therefore, that in designing the main girders or trusses, the center of gravity of the entire moving leaf be coincident with the center of rotation.

In double leaf basculos, at the center of the span a lock called the shear lock is used. The purpose of this lock is to make both leaves of the moving span deflect equally when a greater load is applied to one leaf than to the other. The maximum shear on this lock will occur when one leaf is fully loaded and no load on the other. When the bridge is locked at the center for both moment and shear, the bridge becomes a simple span supported at the live load bearings.

When double leaf spans are designed with a reaction arm or anchor, to resist the overturning force about the live load bearings, due to the live load on the moving leaf, care must be taken in erection to see that the anchor arm does not come in contact too soon. If it does, it causes excessive stresses in the trunnions and their bearings. The anchors should never come in contact until the center of gravity of the combined live and dead loads passes the live load bearings. The anchors furnish a downward reaction, and since the shear lock cannot be counted on for upward reaction (on account of the possibility of full live load on the opposite leaf) the moving leaf, under live load, becomes a beam over-hanging two supports. Before any live load comes on the moving leaf, the center of gravity of the dead load is at the approximate center of trunnion. As the live load moves on to the moving leaf, the center of gravity moves forward toward the live load bearing, and the entire moving leaf is supported at both trunnions and live load or forward bearings, the river arm and counterweight arm overhanging those two supports. As the load continues to move forward, it becomes great enough to balance the entire span, the center of gravity of the combined live and dead loads move to the live load bearings, and the load on the trunnions becomes zero. As the live load increases and continues to move toward the centre of the bridge, the counterweights are over-balanced by the river arm, and the rear anchors come into play, exerting a negative or downward reaction, and the live load bearings take the total load, both live and dead, and a load equal to the negative reaction of the anchor.

Therefore, care should be taken to see that the anchor is so adjusted that it begins to function at the moment that the centre of gravity of the total live and dead load reaches the live load bearings. With this adjustment undue deflection of the trunnions is eliminated. In order that the operation of seating may be quiet, and chattering eliminated, it is customary to provide oak blocks at the anchors, these being slightly resilient, therefore, compress slightly as the load is applied.

In order to know the correct clearance at which to set the anchor blocks, it is necessary to know:-

(1) The difference in deflection of the anchor arm when supported at the trunnion, and when supported at the live load bearings.

(2) The difference in deflection of the trunnion girder at the trunnion supports under no live load on the span and under full live load.

(3) The ratio of the distance from live load bearing to the center of the anchor bracket, to the distance from the live load shoe to the center of the trunnion.

(4) The amount of compression induced in the oak blocks from the full live load reaction.

The clearance between the anchor blocks and the anchor block bracket should be; the rise of the anchor due to the difference in deflection of the trunnion girder under no live load and under full live load; less (a) The difference between the deflection of the counterweight arm with support at live load bearing and with the support at trunnion and (b) The amount by which the oak anchor blocks compress under full live load uplift on the anchors. If this gives a negative result, the oak anchor blocks would be under some compression before any live load came on.

This would prevent a positive seating of the live load shoes on the live load bearings, and, therefore, would cause a chattering of the span. From this it can be seen that it is advisable to so adjust the anchor blocks that they just touch at the instant that the live load shoes come in contact with their bearings.

Particular care should be taken to see that the trunnion bearings are in correct alignment. Inasmuch as the center line of the trunnions will not be at right angles to the final plane of the trusses until the full dead load is on the span, rivetting should not be performed until the entire steel work is in place. This can be held in place by means of temporary bolts. After the steel work in both leaves has been rivetted, except one panel of laterals, these being in place and bolted only, the two leaves should be lowered and checked for alignment, meeting, deflection, etc. If the two leaves are out of alignment, the panel of bolted laterals may be removed and the leaves aligned by means of diagonal rods with turn buckles. The laterals may then be reamed and rivetted. If there is a difference in level between the two adjacent leaves, this may be corrected by means of shims at the live load bearings.

GENERAL DESCRIPTION OF PROJECT

The structure considered in this design is a double leaf trunnion type, bascule highway bridge, with approaches of three spans each, of deck plate girder type.

In the bascule span, there will be two main bascule piers of reinforced concrete, each supporting one bascule leaf and its appurtenant machinery. Each bascule pier also supports one end of the suspended span of the approach.

Each bascule leaf is operated by an electric motor geared to pinions which engage with a rack located at the rear of the counterweight chamber, and in a vertical plane passing through the center line of the moving leaf and parallel to the main longitudinal girders. There is one motor for each leaf, directly connected to the operating machinery. When both moving leaves are lowered for traffic, they are locked at the center of the bridge by means of shear locks. When in the raised position, the roadway is protected by means of electrically operated gates. The operation of the centre locks is interlocked with the roadway gates, and the operation of raising the moving leaves is interlocked with the operation of the center locks, making it necessary to lower the gates before the center locks can be drawn, and then to draw the center locks before the moving leaves can be raised.

Both leaves are provided with automatic electric devices to break the electric circuit and to operate electric brakes when the moving leaf is near the end of its travel. A gasoline engine, together with hand brakes is provided for emergency operation. The center locks are operated by means of a small electric motor, direct connected to gears pinions and crank-shaft located under the deck at the center of the bridge. Provision is made for hand operation in case of emergency.

Each moving leaf will be balanced with a counterweight. These counterweights are of reinforced concrete construction, built o and around a structural steel frame. Each counterweight has within i two chambers for the accommodation of balance blocks to provide for adjustment for variation in weight due to seasonal variation, snow, ice, etc.

The bascule leaves are of the deck girder type. Floor beams are plate girders and stringers of steel I beams. The floor system of the moving leavings will be of creosoted plank with asphalt plank wearing surface.

Each counterweight pit is equipped with a two inch centrifugal sump pump operated by a three H.P. motor to drain off leakage through the walls or from the roadway above.

Approach spans are of the deck plate girder type concrete encased. Each approach consisting of three spans designed in the form of arches. The center span consists of a beam over-hanging the two center piers. On the cantilever ends a span is suspended with a pin connection, the other end of this suspended span being supported on the bascule pier or shore abutment as the case may be.

Approach span decks are of reinforced concrete, supported on steel I beam stringers, which in turn are frames into plate girder floor beams. It should be noted here that in the pin connected panel the stringers are to have flexible connections to the floor beams. All floor beams and stringers are to be encased with concrete, while the main girders are to be wrapped with galvanized steel fabric and gunited.

After erection, all exposed metal parts of the bascule span are to be painted to match the concrete portions of the balance of the structure, the intention being to produce a bridge having the beauty and grace of a multiple span arch bridge, and at the same time the utility of a bascule lift.

Both bascule piers are protected with fenders. This protection is required by the U. S. War Department on all bridges constructed across navigable rivers of the United States. These fenders are built up of 12 x 12 creosoted waling, bolted to pile dolphins with galvanized bolts. Each dolphin is built up of three pipes, except the dolphins at each intersection of the pier protection and wing fender, which is a seven pile dolphin.

Too much emphasis cannot well be put on the importance of exercising good judgement and experience in the construction of pier fenders. The piles must have sufficient resiliency to give under the impact of shipping, and the waling must be of sufficient strength to transmit the shock of impact to the piling without breaking. It is certainly poor economy to build a pier fender of flimsy construction that may result in an accident to the pier, and may even cause the collapse of the bridge itself. No fender at all is far better than one of light or insufficient construction, for with no fender, care would be exercised to avoid accidents, while the fender of inadequate construction would only create a false sense of security.

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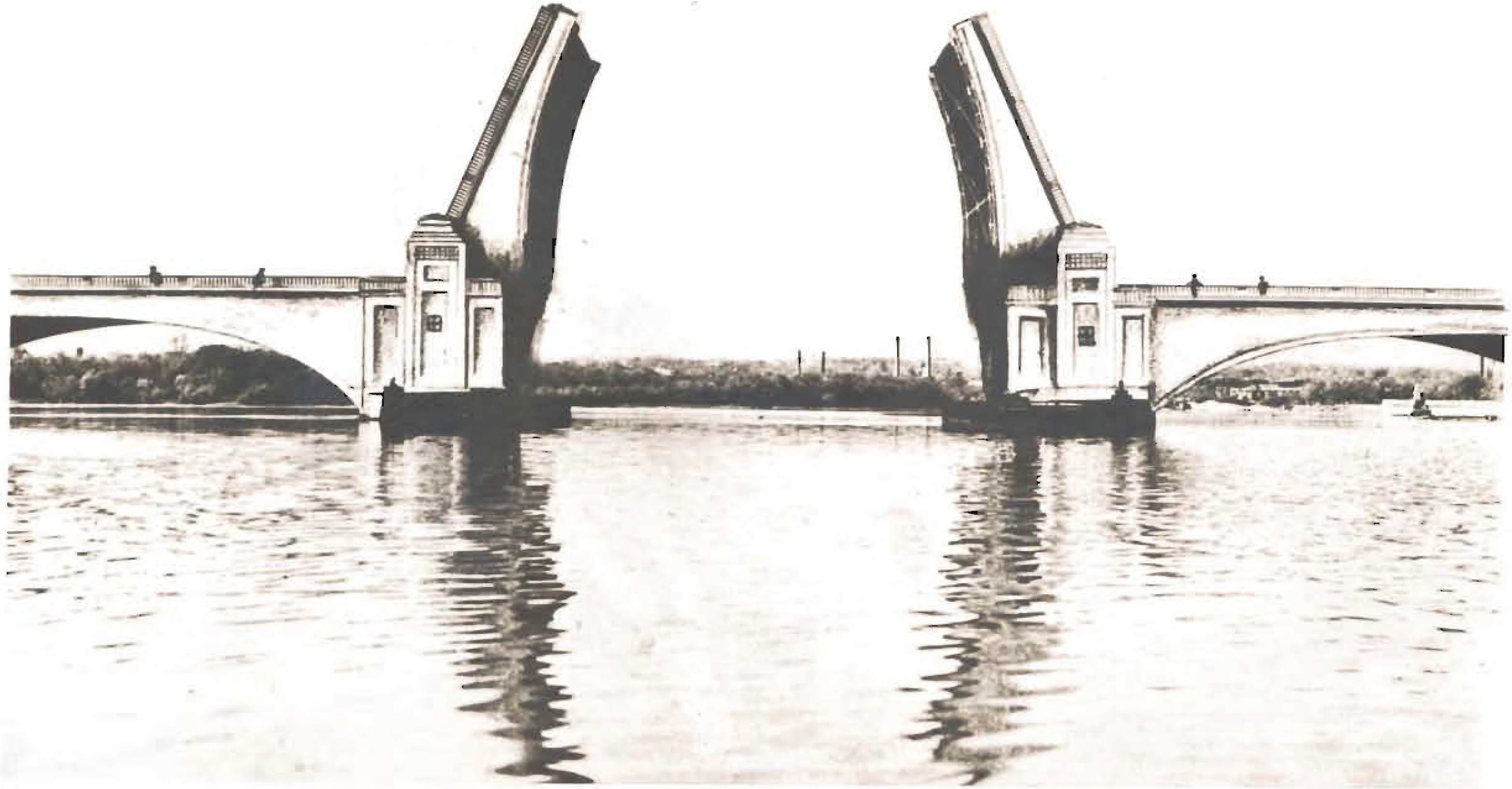


Plate #1

GOVERNING DIMENSIONS

Clear Channel at water line	100'-0"
Clear Channel at tip of raised leaves (L of opng.)	=
Elevation of roadway surface	100.00
Low water level	76.00
High water level	86.00
Bottom of existing channel	55.00
" " proposed "	50.00
Bottom of piers	50.00
Clear width of roadway between curbs	24.00'

LOADS

Dead Load	Asphalt Board	85#/cu.ft.
" "	Plain timber	4#/Bd.ft.
" "	Creosoted "	5#/ " "
" "	Concrete	150#/cu.ft.

LIVE LOADS

2-15 Ton Trucks side by side followed by a uniform load of 80#/□' of bridge floor
2-15 Ton Trucks of following dimensions:-
Distance between axles 10ft., wheel guage 6'
Two thirds of the load on the rear wheels
Space occupied by Truck 20ft. long by 9ft. wide

IMPACT

In order to provide for the dynamic increment of stress due to the moving loads, the computed Live Load stresses are increased by the following percentages:-

Floor stringers	60 percent
Intermediate floor beams	50 " "
Stringers & floor bms. at break in floor	100 " "

The maximum live load stresses in the longitudinal girders are increased by a percentage obtained by the formula P(percentage) $= \frac{100}{2.6L+300}$, where L = the loaded length of bridge, in feet, producing the max. stress in the member. When the bridge is open or in motion, the dead load stresses are increased by a flat 20 per cent to provide for vibration effects.

IMPACT AT ANCHORAGE
The computed uplift stresses in the anchorage at heel of anchor arm are to be increased by 40 per cent

WIND LOAD
When in the closed position the bridge is designed for a lateral load of 30#/□' of exposed surface
When in the open position a wind force of 15#/□' is considered as acting in any direction. The stresses caused by the above specified lateral forces are not increased for impact by any percentage such as given by the formula given in the previous paragraph.

For stresses produced by longitudinal or lateral wind forces combined with those from live load, dead load and impact stresses, the allowed unit stresses may be increased 25 per cent over those given in the table of allowed stresses, but the section shall not be less than would be required if the wind forces were neglected and the 25 per cent increase not allowed.

SHEAR AT CENTER OF BRIDGE
The amount of shear transferred from one leaf to the other through the centre lock is assumed to be the amount given by the following formula:-

$$S = \frac{P}{4}\left(\frac{A}{L}\right)^2\left(3 - \frac{A}{L}\right)$$

Where S = Shear carried by centre lock
P = any concentrated load
A = distance of P from Live Load girder shoe
L = length of forward arm (centre of bridge to live load girder)

PERMISSIBLE STRESSES
The size and make-up of each member shall be proportioned for stresses due to the following loads, and combinations of loads:-
(1) Dead load only
(2) Dead load, live load and impact
(3) Dead load, live load and impact, together with wind loadings. For these various load combinations, the permissable unit stresses in the following table shall not be exceeded

Kind of Stress	Permissable Stress for Combined loads, dead, live and impact (lb per sq. inch	Permissable Stress for Dead Load Only (lb per Sq. Inch
Steel reinforcement in tension	18000	14000
Steel in compression, 15 times stress in surrounding concrete		
Steel in Shear	12000	9000
Concrete in tension	0	0
Concrete in compression due to bending	650	500
Concrete in bearing	400	300
Concrete in shear (no web reinforcing)	40	30
Concrete in shear (diagonal tension) beams having shear reinforcement of bent-up bars and stirrups	120	90
Bond of concrete & deformed bars	100	70

Counterweights - The counterweights shall be so designed that they will balance the moving part in all positions and so fashioned that they can be easily and properly adjusted for variation in weight by adding or removing definitely located weights. They shall be of concrete construction, built on and around a structural frame thoroughly reinforced and hooped, or else encased in steel boxes.

Anchor Arm Lateral System. The lateral bracing will be designed to extend to the trunnions as well as to the anchorage at the heel of the counterweight and will be proportioned to carry the full load to either point.

Buffer Blocks Both the anchor buffer blocks and the blocks for stopping the bridge in the open position are to be made of sound, well-seasoned white oak, entirely free from knots or other imperfections.

Other details of design and fabrication are according to the ordinary rules for such work as found in the various specifications for the design and fabrication of steel bridges.

Kind of stress	Permissable Stress for combined loads, dead, live and impact (lb. per sq. inch	Permissable Stress for Dead Load only (lb. p sq. inch
Axial and bending tension on net section of:-		
Structural steel	16000	12000
Upset bars and rods where not annealed	12000	9000
Axial compression on gross section+:-	$16000-70\frac{L}{r}$	$12000-50\frac{L}{r}$
(L = unsupported length of member in inches)		
Direct compression on:-		
Cast-steel bearing and struct. steel plates	16000	12000
Cast-iron blocks	14000	10000
Bending on:-		
Extreme fibre of pins	24000	18000
Shearing on:-		
Rivets and turned bolts in floor connections, shop and field	8000	6000
Pins and shop rivets, except in floor conn	12000	10000
Turned bolts and field rivets except in floor connections	10000	8000
Web of girders, net section	12000	10000
Web of girders, gross section	10000	8000
Bearing on:-		
Rivets and turned bolts in floor connections, shop and field	16000	12000
Pins and shop rivets, except in floor conn.	24000	20000
Turned bolts and field rivets, except in floor connections	20000	16000
Expansion rollers, per lin. inch (d = diameter of the roller in inches)	500d	500

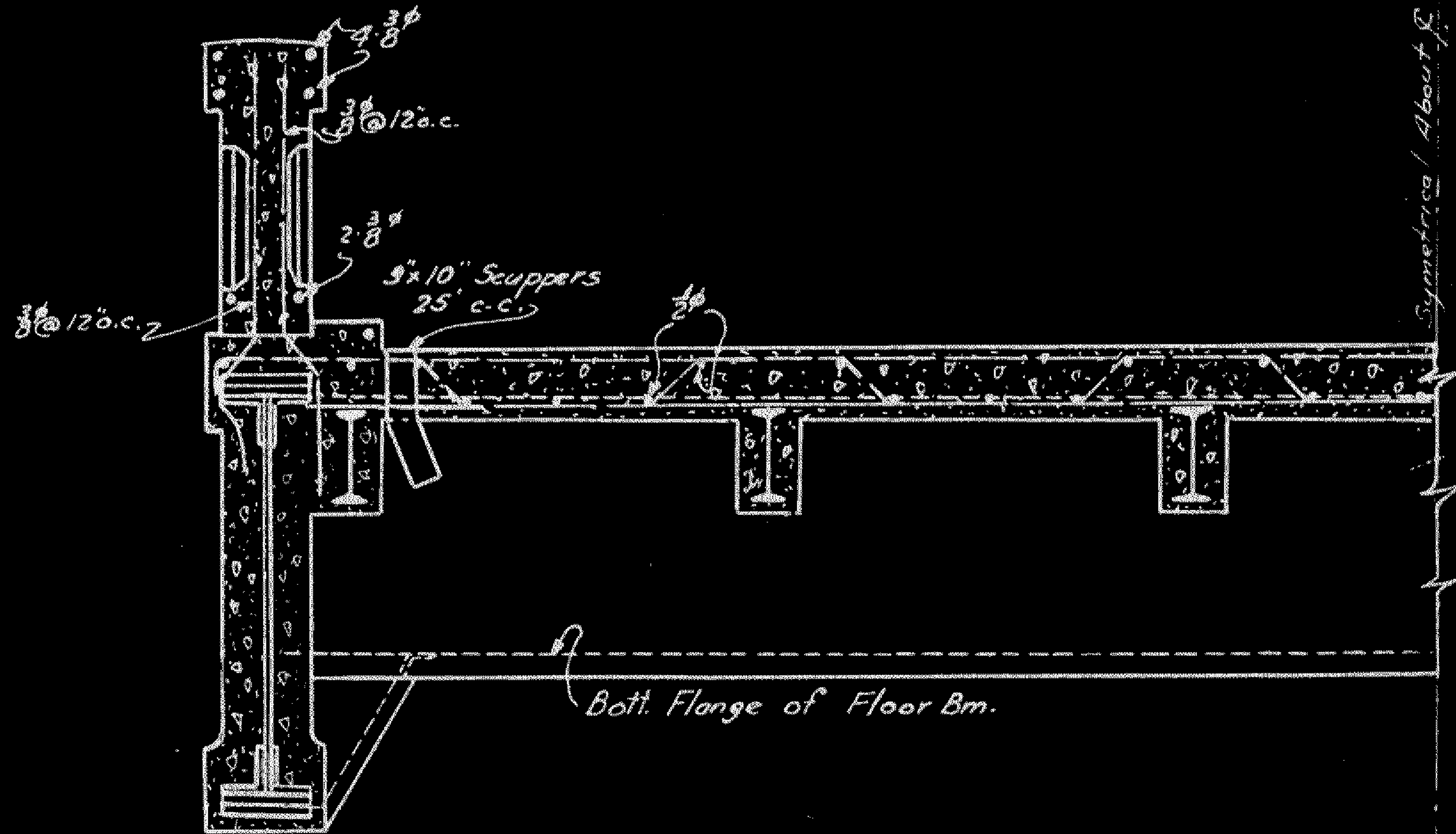
+) $\frac{L}{r}$ shall not exceed 100 for main members nor 120 for subordinate members

Allowable Static Pressure on Masonry

Unreinforced mass concrete may be loaded as follows

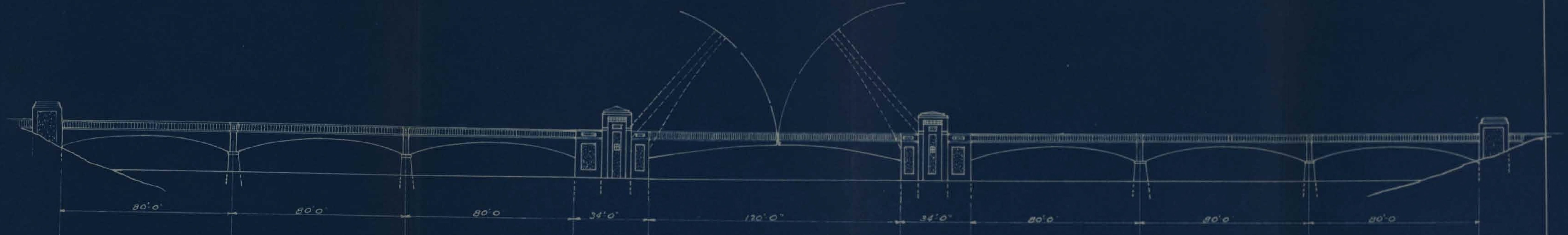
1:2½:5 Concrete not more than 500#/□"

1:2:4 Concrete not more than 650#/□"

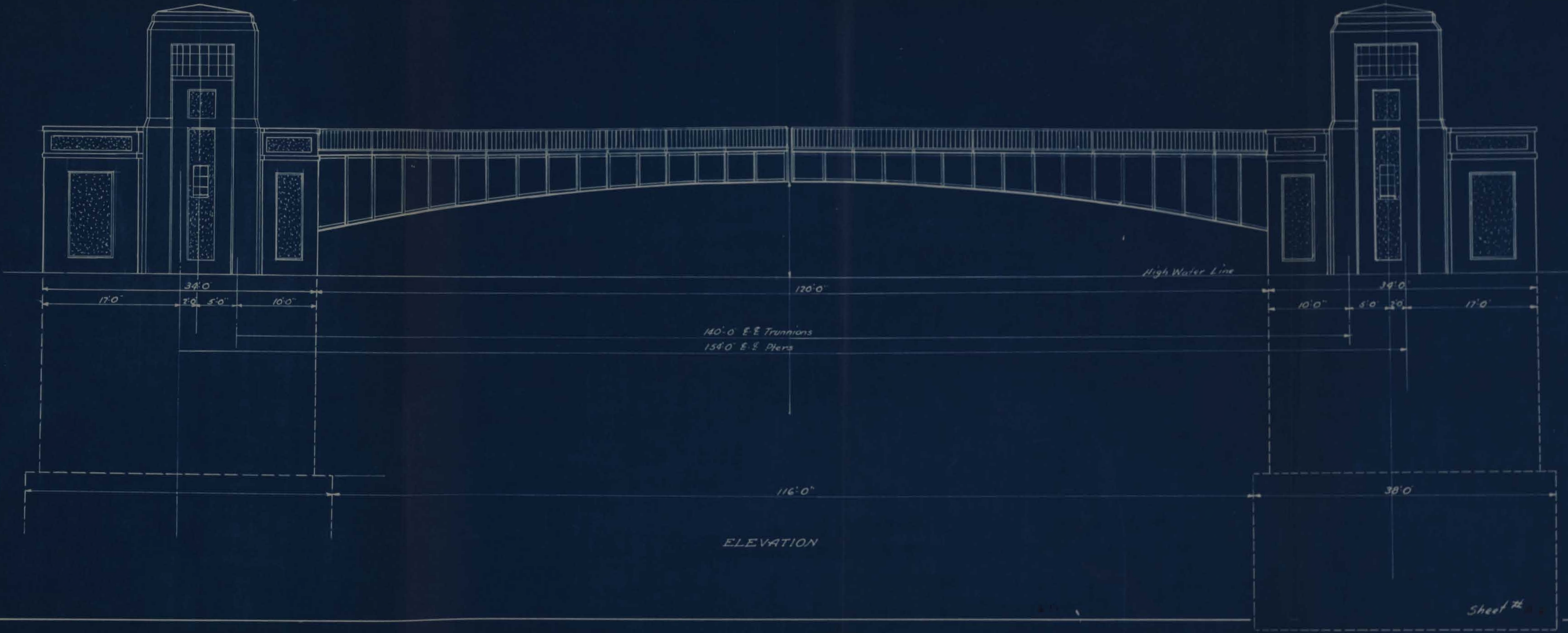


TRANSVERSE SECTION
THRU. APPROACH DECKS

Sheet #



LONGITUDINAL ELEVATION



ELEVATION

Sheet #

GENERAL DESIGN Sheet #B5

Stringers and Floor Beams

Stringers 15 ft. Span

Spaced @ 2.42' apart

Dead Loads	$1\frac{1}{2}$" asphalt board $\frac{1.5}{12} \times 85$	=	10.65
" "	4" creosoted pine flooring	=	20.00
" "			30.65 #/ft
" "	wt. of stringer		40.00 #
	30.65×2.42	=	74.40
Total dead load per ft. of stringer			114.40

say 115 #/ft.

			Shears
Dead Load Mom.	$\frac{115 \times 15 \times 15}{8}$	= 2590 ft.#	1020
L.L. "	$= \frac{10000 \times 15}{4}$	= 37600 ft.#	5000
Impact	60% of 37600	= 22600 ft.#	3000
		62790 ft.#	9020

Section Modulus Req'd. $\frac{62790 \times 12}{16000} = 47.2$

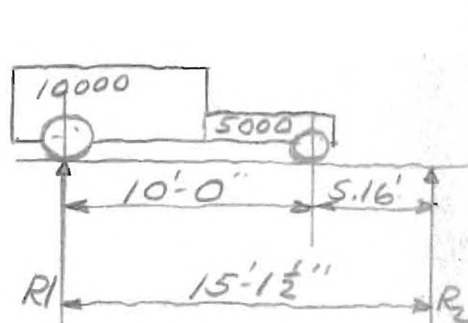
A 12" I @ 45# will be adopted.

Floor Bm

F.B.3

$\frac{5000 \times 5.16}{15.16} = R_1\ 1700$

$R_1 = 1700 + 10000 = 11700\#$



Dead Loads

$1\frac{1}{2}$" asphalt	$= 1 \times 15.16 \times 10.65$	=	162
Flooring	$= 1 \times 15.16 \times 20$	=	304
Stringers	$= 15.16 \times 45 \times \frac{12}{29.5}$	=	278
Floor Bm. 3		=	135
Total dead load		=	879 #/ft.

See diagram on next sheet

$M_d = M_a = 35000(5.165) - 879\left(\frac{5.165}{2}\right)5$

$= 182000 - 11350 = 170{,}650$

$M_b = 35000 \cdot 11.165 - 11700(6) - 11.165(879)\left(\frac{11.165}{2}\right)$

$= 392000 - 70200 - 54600 = 267{,}200$ ft.#

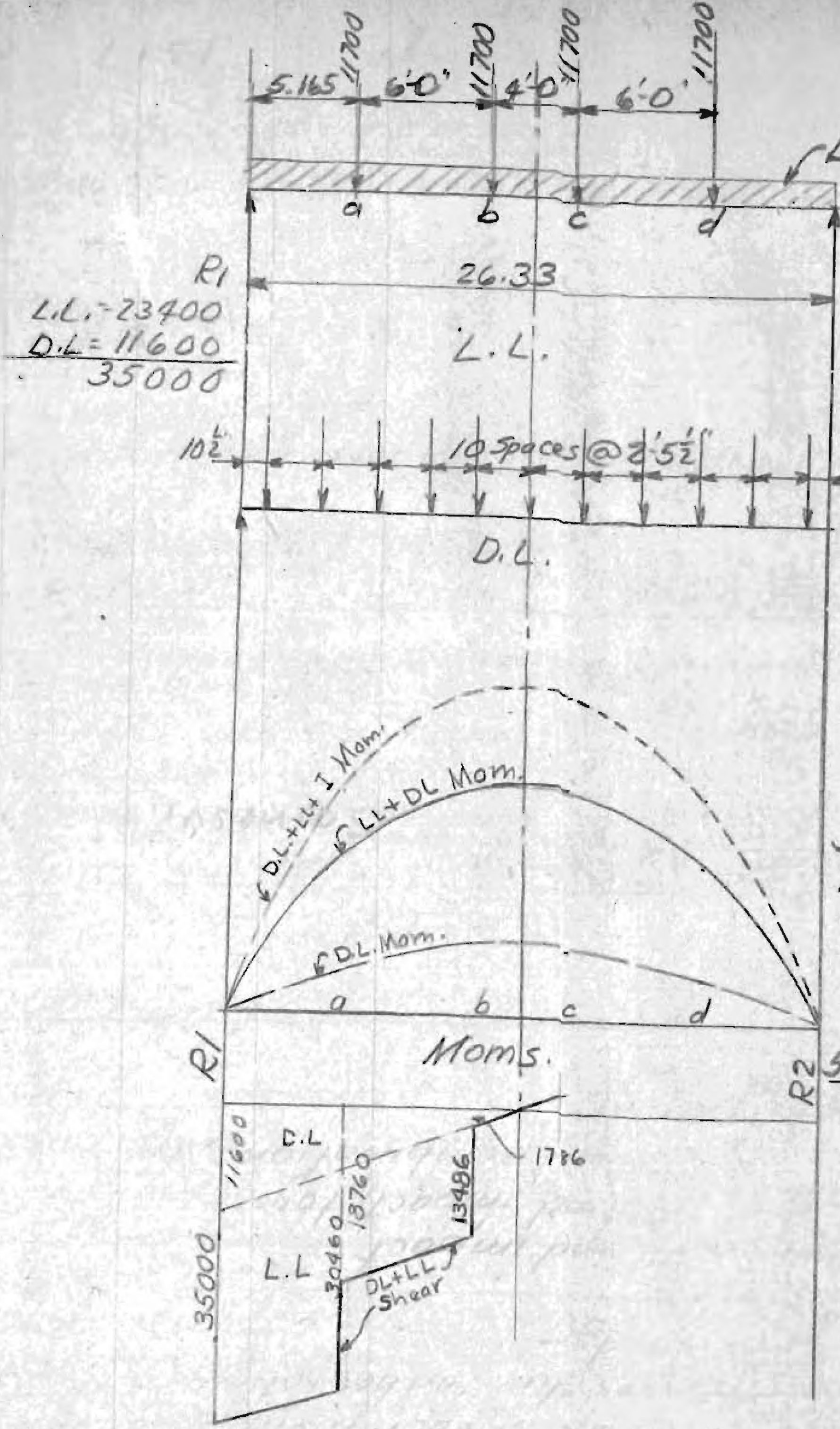
Shears Lt. of pt. a $= 35000 - (879 \cdot 5.165) = 30460\#$

Rt. " " a $= 30460 - 11700 = 18760\#$

Lt. " " b $= 18760 - (879 \times 6) = 13486\#$

Rt.

Sheet #B-6



D.L. 879 #/ft.

R2
L.L = 23400.
D.L = 11600
35000

D.L. Mom
$= \frac{879 \times 26.33 \times 26.33}{8}$
= 76300 ft. #

L.L. Mom
= 267200 − 76300
= 190900 ft. #
50% I 95450
Total = 286350 ft. #

Shears

L.L	=	23400#
D.L	=	11600
50% I	=	11700
Total	=	46,700
Say		47000#

Web req'd. at 10000 #/□" $= \frac{47000}{10000}$ = 4.7□"
Web used = 19.88□" Gr.
Eff. depth 4'-5¼" = 4.25'
Flange stress = 67400#
Section Req'd. (net) = 4.23□" net

	Gr	Gr	Net
⅛ Web	2.48	2.48	2.48
Top Flange 2Ls 4×4×7/16	6.62		
Bott. Flange 2Ls 6×3½×7/16		7.94	7.06
Total	9.10	10.42	9.54

Floor Bms. FB1 & FB2

Shears & Moms. same as in FB3

Web req'd. $= \frac{47000}{10000}$	$= 4.7^{\square}$		
Web used $31 \times \frac{3}{8}''$	$= 11.62^{\square}$ Gr.		
Eff. Depth	2.42 ft.		
Flange Stress	119000 #		
Sect. Req'd (Net)	$7.42^{\square}$ net		

	Gr.	Gr.	Net
1/8 Web	1.45	1.45	1.45
Top Flange 2Ls $4 \times 4 \times \frac{5}{8}''$	9.22		
Bott. " 2Ls $6 \times 4 \times \frac{1}{2}''$		9.50	8.50
Total	$10.67^{\square}$	$10.95^{\square}$	$9.95^{\square}$

Floor Bm. FB0.

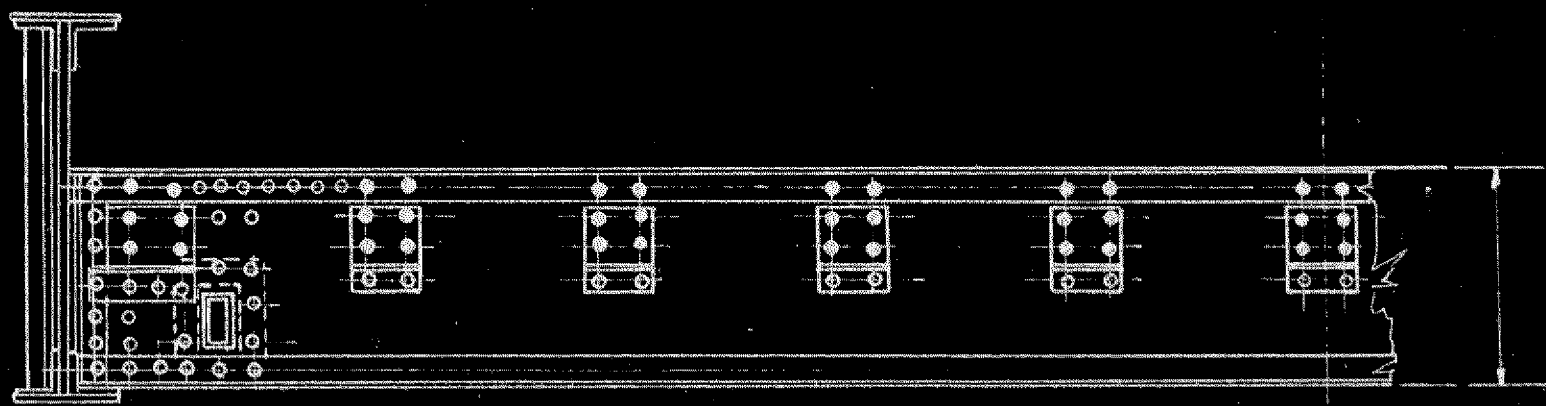
D.L. Mom. = 72000 ÷ 2	= 36000 ft.#
L.L. Mom.	= 190,900 "
100% Impact	= 190,900 "
Total Moment	417800 " "
D.L. Shear 11600 ÷ 2	= 5800 #
L.L. "	= 23400 #
100% Impact	= 23400 #
	= 62600 #

Shear Transferred from one leaf to the other through centre lock

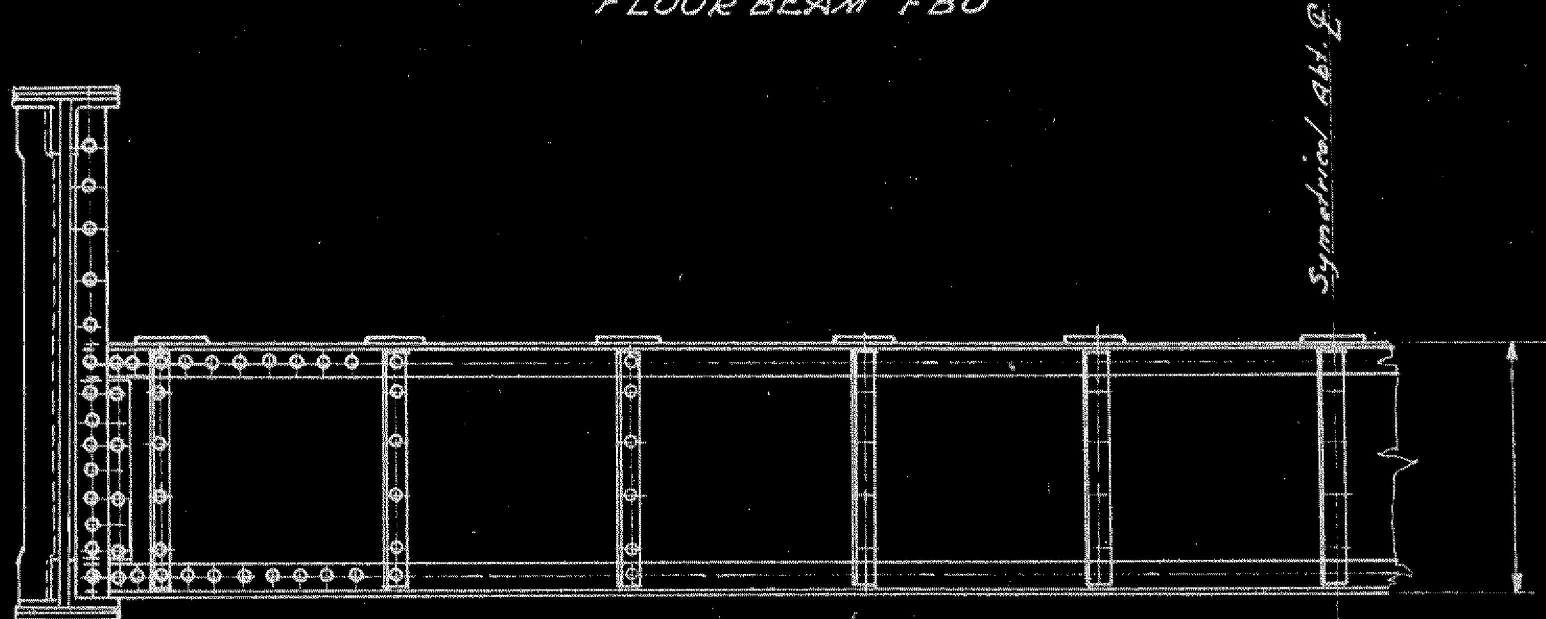
$$S = \frac{P}{4}\left(\frac{A}{L}\right)^2\left(3-\frac{A}{L}\right) = \frac{20000}{4}\left(\frac{60.3}{60.3}\right)^2(3-1) = 10000\# \quad = 10\,000\#$$

Total Shear	= 72600 #
Web Req'd. = 72600 ÷ 10000	= $7.26^{\square}$ Gr.
Web Used $= 27 \times \frac{3}{8}''$	= $10.125^{\square}$
Eff. Depth = 2.08 ft.	= 2.08 ft.
Flange stress	= 205,000 #
Sect. Req'd. Net	= $12.55^{\square}$

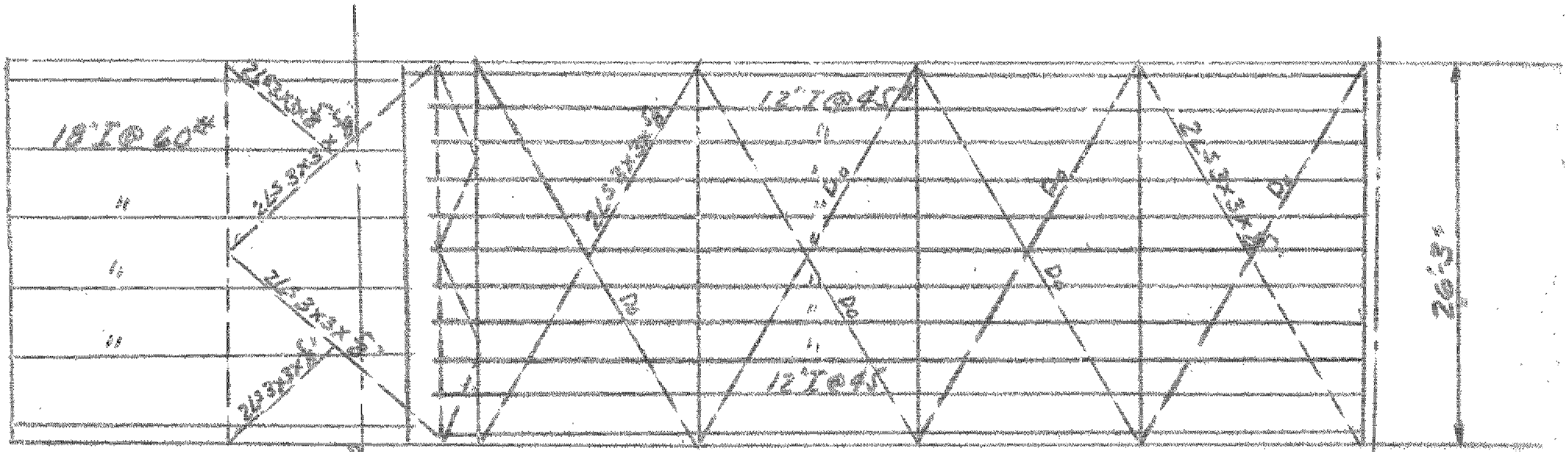
	Gr.	Gr.	Net
1/8 Web	1.27	1.27	1.27
Top Flange 2Ls $6 \times 6 \times \frac{13}{16}$	18.18		
Bott " 2Ls $6 \times 6 \times \frac{13}{16}$		18.18	17.18
Total	19.45	19.45	18.45



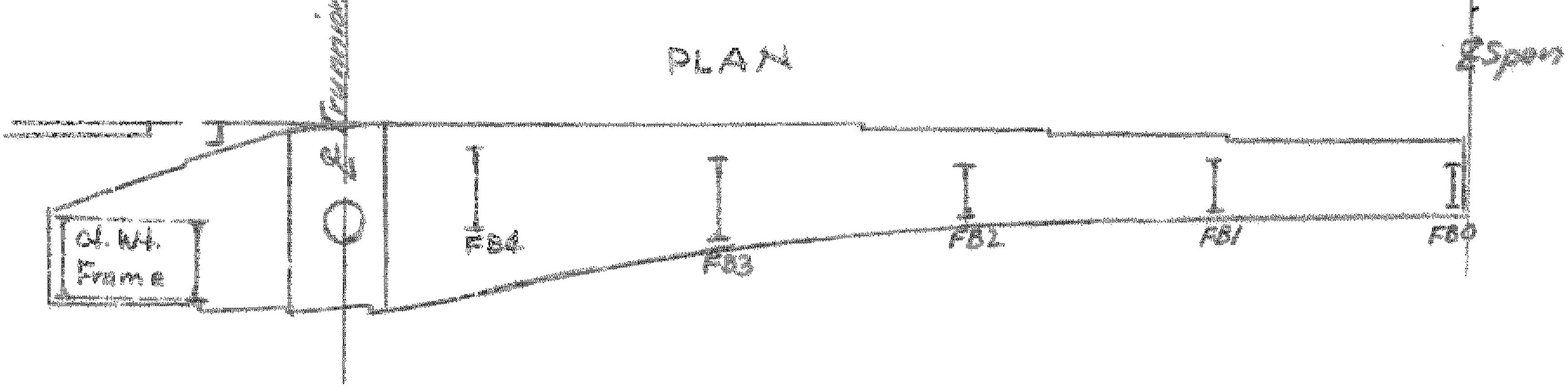
FLOOR BEAM FB0



FLOOR BMS. FB1 & FB2

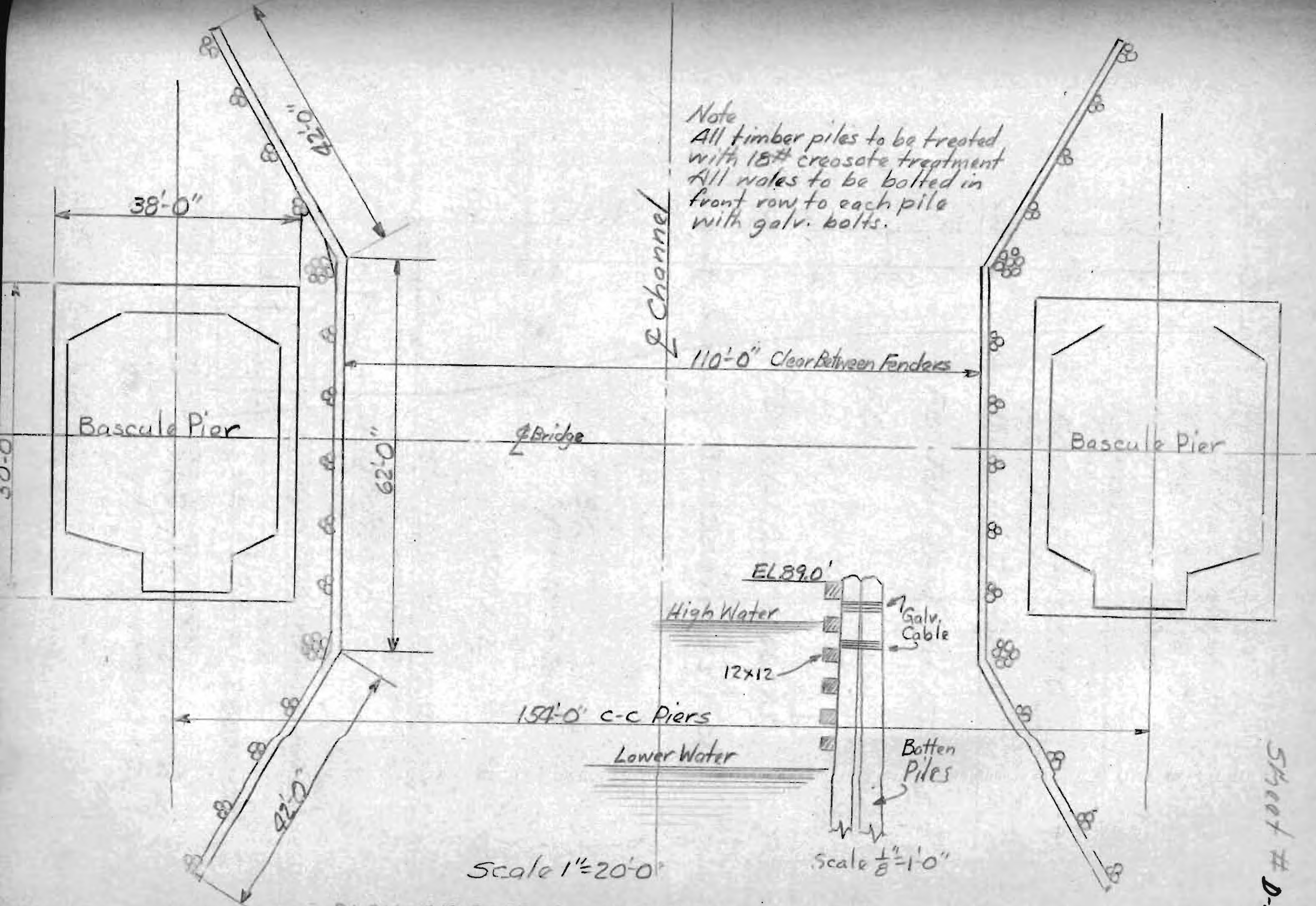


PLAN



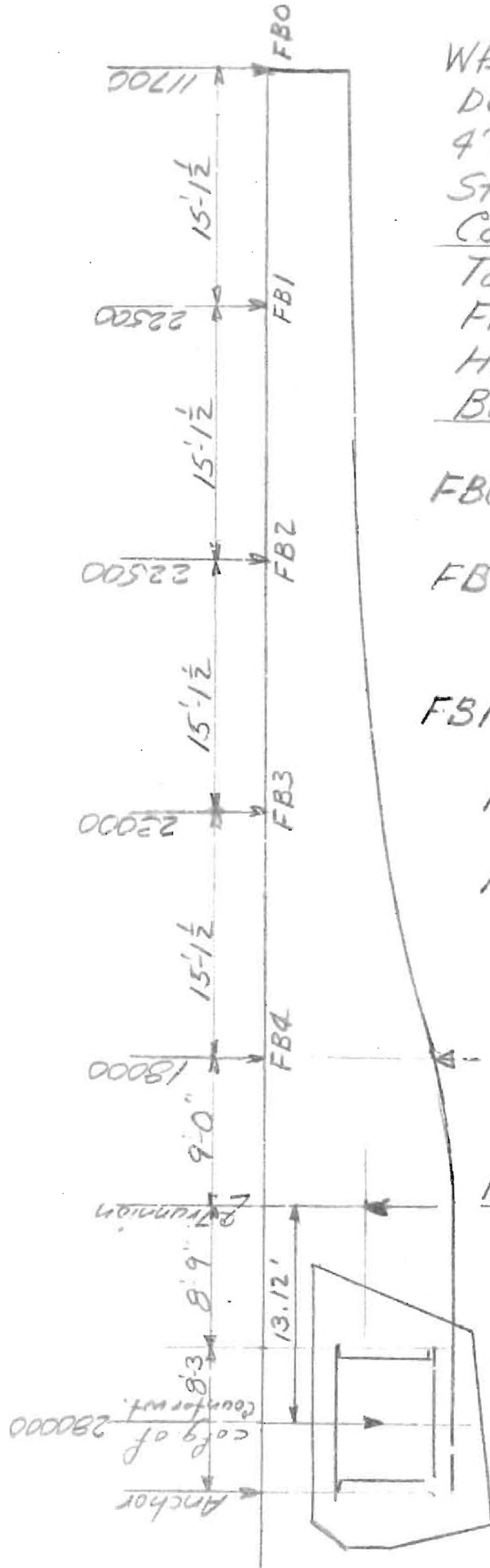
ELEV. OF MAIN GIRDER

Sheet No. D-14



PLAN OF PIER & FENDER LOCATION

Sheet # D-10



Wts. of F.B.O

Dead loads Asphalt 18.75×13	= 244
4" Pine flooring @ 5#/Bd. ft. 5×4×13	= 260
Stringers 5.5×40	= 220
Connections	= 40
Total	= 764
FBO 226×13.2÷7.6	391
Handrail	100
Bascule Girder	300
	1555

FBO Total Deadload 1555×7.5 = 11663

FB1	764
391+100+245	736
	1500

FB1 Total D.L 1500×15 = 22500

FB2 Same as FB1 = 22500

FB3 = 23000

FB4	764
100+380+265	745
	1509

1509×12 = 18108

$$\frac{11700(70)+22500(54.875)+22500(39.75)+23000(24.625)}{13.12}+$$

$$\frac{18000(9)}{13.12}=P_{cwt.}$$

$$\frac{819000+1{,}230{,}000+894000+567000+162000}{13.12}=P_{cw}$$

$$=\frac{3672000}{13.12}=280000\#$$

∴ Weight of counterweight and supporting frames = 280000#

Weight of Counterweight Frames

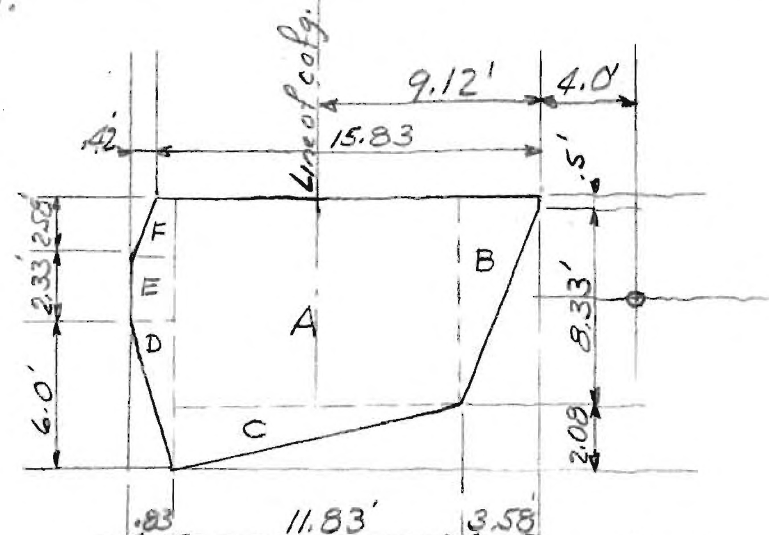
2 Frames Thus

Girders	℄ 79×13	=	1027
	4 Ls	=	79
	2 ℄s	=	39
	Connections	=	18
			1163

2×1163×13 = 30280#

Volume of steel $= \frac{30280}{495} = 61.8$

62×150 = 9300# equiv. weight of concrete displac
by steel.



Approx. Typical Counterweight Section

Design of counterweight

Volume of prism 10.92×26.33×16.25 = 4673cu

Volumes to be deducted less = 1285

3388

2(4.25×3.25×6.33)	=	175
(3.58×8.33×26.33)÷2	=	392
(6×.83×26.33)÷2	=	66
(.42×2.58×26.33)÷2	=	14
6×15.83×.83×1.5	=	118
3.58×2.08×26.33	=	196
(11.33×2.08×26.33)÷2	=	324
		1285

3388×150 = 510000

Center of gravity

Areas (A)	11.83×8.83 =	= 104.5
B.	(8.33×3.58)÷2	= 14.9
C.	(11.83×2.08)÷2	= 12.3
D.	(6×.83)÷2	= 2.5

Center of gravity of Counterweight Ctd.

E.	2.33 × .83	= 2
F.	(.83 × 2.6) ÷ 2	= 1
Total Areas		= 137.2□'

Area Moments.

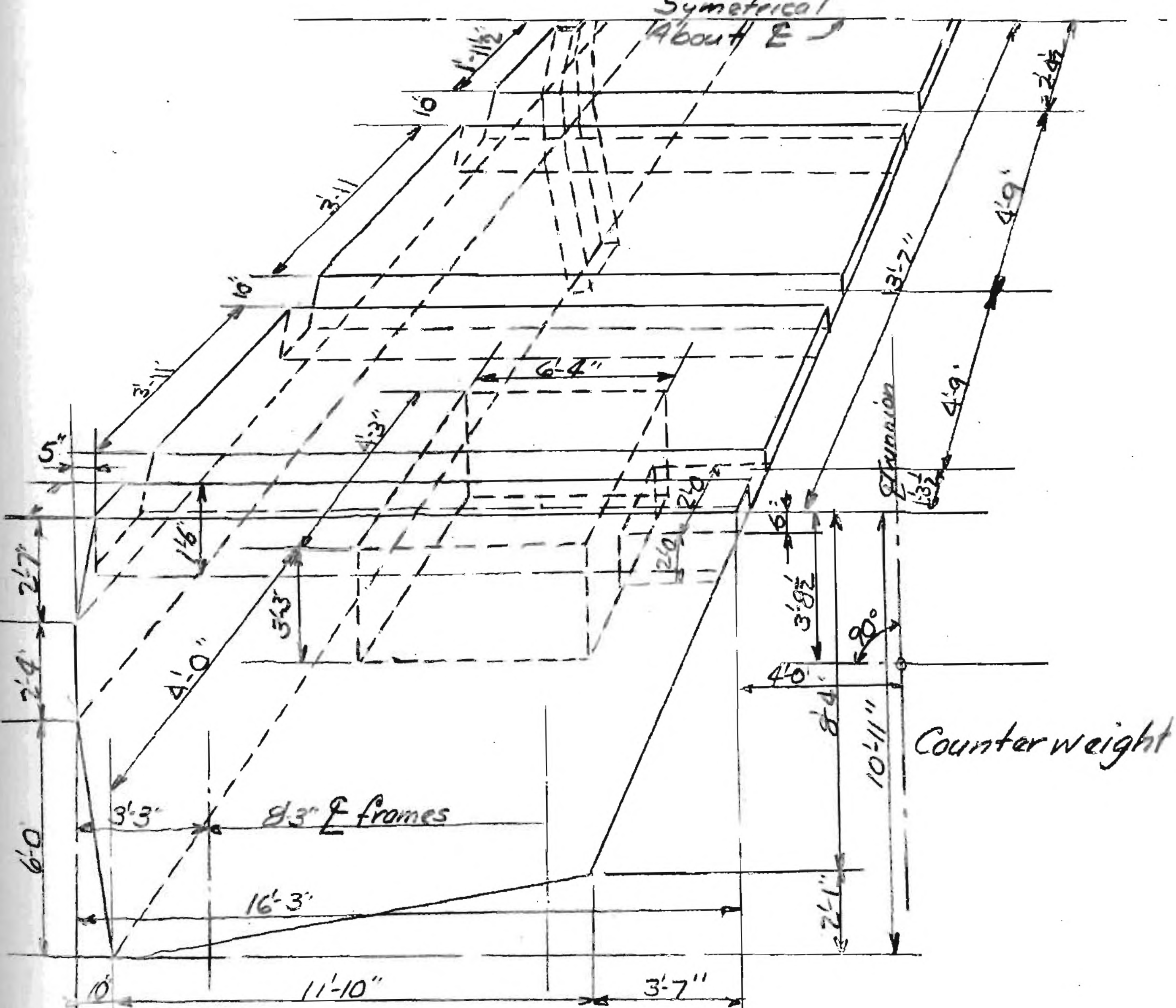
A.	104.5 × 9.49	= 990.00
B.	14.9 × 2.5	= 37.25
C.	12.3 × 11.3	= 138.99
D.	2.5 × 15.7	= 39.25
E.	2 × 15.8	= 31.60
F	1 × 16	= 16.00
		1253.09

$\frac{1253.09}{137.2} = 9.12'$

See Sketch

Due to seasonal variation counterweight balance blocks will be provided to the extent of approx. 2½ per cent of the entire weight of counterweight

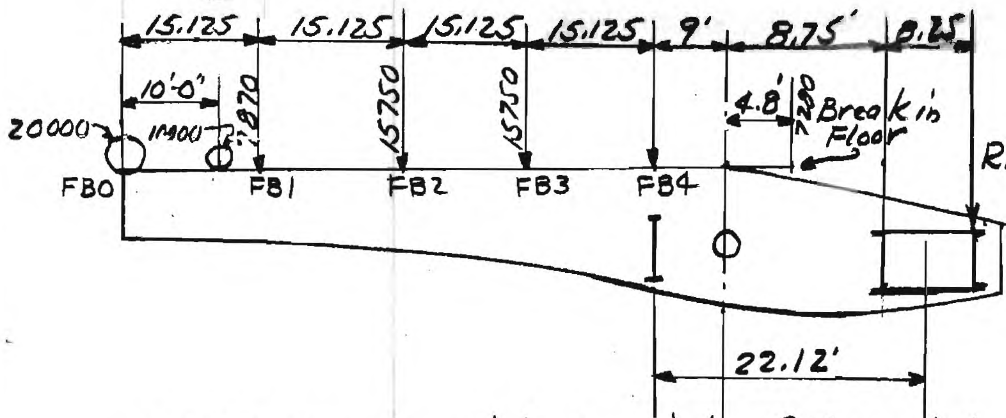
Volume of counter weight chambers =
4.25 × 6.33 × 3.25 = 87.5 cu. ft. each (2 Thus)
87.5 × 150 = 13110 #
Using blocks equiv. to one cu. ft. ea.
88 × 2 = 176 Blocks req'd. for each counterweight



Symeterical About ℄
1'-11½"
10"
3'-11"
10"
3'-11"
2'-6½"
4'-9"
3'-7"
4'-9"
6'-4"
4'-3"
5"
1'-6"
2'-0"
2'-0"
℄ Trunnion
1'-1"
1'-3½"
6"
3'-8½"
90°
4'-0"
5'-3"
4'-0"
2'-7"
2'-4"
6'-0"
8'-4"
10'-11"
Counterweight
3'-3"
8'-3" ℄ frames
16'-3"
2'-1"
10"
11'-10"
3'-7"

Sheet #B12

Design of Main Bascule Girders



2-15 Ton Trucks side by side followed by uniform load of 80#/□'

20000# for rear wheels per girder

10000# for front wheels per girder

80×13 = 1040# per ft. of girder

1040×15.125 = 15750 # per panel

L.L. Shears		FB0 - FB1	=	30000#
L.L	"	FB1 - FB2	=	45750#
L.L	"	FB2 - FB3	=	61500#
L.L	"	FB3 - FB4	=	77250#

L.L Moments

20000×60.5+10000×50.5+15750×37.812+15750×22.687+ 15750×7.562 = 1,210,000+505000+596000+358000+119200 = 2,788,200 ft. # = Max L.L. Mom. = 2,788,200 ft.#

L.L. Mom. at FB1 = 20000×15.125+10000×5.125 = 353000 ft.#

L.L. " " FB2 = 20000×30.25+10000×20.25+ 15750×7.56 = 926,500 ft.#

L.L. Mom. at FB3 = 20000×45.375+10000×35.375+ 15750×22.687+15750×7.56 = 1,732,375 ft.#

L.L. Mom. at front Counterweight frame 36000×8.25 = 297000 ft.#

D.L. Moms. ℄ Trunnion 11700×70+22500×54.875+22500×39.75+ 23000×24.125+18000×9 = 3,650,000 ft.#

D.L. Mom.	F.B.1	11700×15.125	= 172000"
D.L. "	F.B.2	11700×30.25+22500×15.125	= 694000"
D.L. "	F.B.3	11700×45.375+22500×30.25+ 22500×15.125	= 1,550000"

D.L. Mom. F.B.4 $11700\times 60.5 + 22500\times 45.375 +$
$22500\times 30.25 + 23000\times 15.125$ = 2,636,000 ft

D.L. Mom. Rear of Cwt. frame 4×280000 = 1,120,000

Pivoting Dead Loads at FB4

D.L. Mom. at FB4	= +2636000 ft
L.L. " " FB4	= +2788000 "
Total " " FB4	= +5424000 "

$$RA = \frac{5{,}424{,}000 - 280000\times 28.12 - 13600\times 9}{26} = 99300\#$$

Anchorage Reactions

Fulcrum at FB4 = L.L. bearing

+L.L $20000\cdot 56.36 + 10000\cdot 46.36 + 7560\cdot 45.3 +$
$14500\cdot 30.8 + 14500\cdot 15.1$ = 2,600,600

+D.L. $11700\cdot 61 + 22500\cdot 45.3 + 22500\cdot 30.18 +$
$22500\cdot 15.09$ = 2,752,000

+ 5,352,600

−D.L & −L.L

$-10000\cdot 9 - 280000\cdot 22.12$ − 6,290,000

+ 937,400

$937400 \div 26 = 36000$ = Shear at rear Ct. Wt. Frame

$937400 \div 17.75 - 36000 = 16800$ " " front " " "

See tabulation of stresses in longitudinal girders for design of main girders.

Design of splices in longitudinal Girders Pt. 2

Dist. back to back of flange Ls = $60\frac{1}{4}$"

Clear dist. between vert. legs = $60\frac{1}{4}" - 6\frac{1}{8} - 6\frac{1}{8} = 48$"

Allowing $\frac{1}{8}$" at top & bott. of splice plates leaves $47\frac{3}{4}$" as hgt. of splice plates

Area of web = $60\times\frac{1}{2} = 30$ sq. inches

$30\times .75 = 22.5^{\square"}$ $\therefore$ $22.5 \div 47.75 = .47" = \frac{1}{2}"$

Assume rivets @ 3" spacing K = 9600 = shear value of $\frac{7}{8}$" rivet, $r_0 = 21$"

$M_r = \frac{K}{r_0}\Sigma r^2$, $M_r = \frac{9600}{21}\times(3^2+6^2+9^2+12^2+15^2+18^2+21^2)$

Resisting Mom. of rivets $= M_r = \frac{9600}{21}\times 10080 = 4{,}930{,}000$ in #

Splice between Pt. 3 & Pt. 4
Distance back to back of flange ∠s = $97\frac{1}{4}''$
Clear distance between vertical Legs $= 97\frac{1}{4} - 12\frac{1}{4} = 85''$
Allowing $\frac{1}{8}''$ at top and bott. of splice Pls. leaves $84\frac{3}{4}''$
Area of web $= 97 \times \frac{9}{16} =$ $54.6^{\square}$
$54.6 \times .75 = 41^{\square''}$ ∴ $41 \div 84.75 = .484 = \frac{1}{2}''$
Spacing rivets $2\frac{1}{4}''$

$$M_r = \frac{9600}{29.25} \times \left[8(\overline{4.5}^2 + 9^2 + \overline{13.5}^2 + \overline{16}^2 + \overline{18.25}^2 + 16(\overline{20.5}^2 + \overline{22.75}^2 + \overline{25}^2 + \overline{27.5}^2 + \overline{29.25}^2)\right]$$

$$M_r = 328\left[8(20.3 + 81 + 182.5 + 256 + 333) + 16(425 + 518 + 625 + 755 + 858)\right]$$

$$M_r = 328\left[(872.8)8 + 16(3181)\right] = 328(6980.4 + 50896) = 328 \times 57876 = 18983000 \text{ in.}\#$$

Pitch of flange rivets, max vert. shear $= 453000\#$

Value of rivet $= 9600\#$
Eff. depth $=$ 107 inches

$$\frac{9600 \times 107.75}{453000} = 2.28 = 2\frac{1}{4}''$$

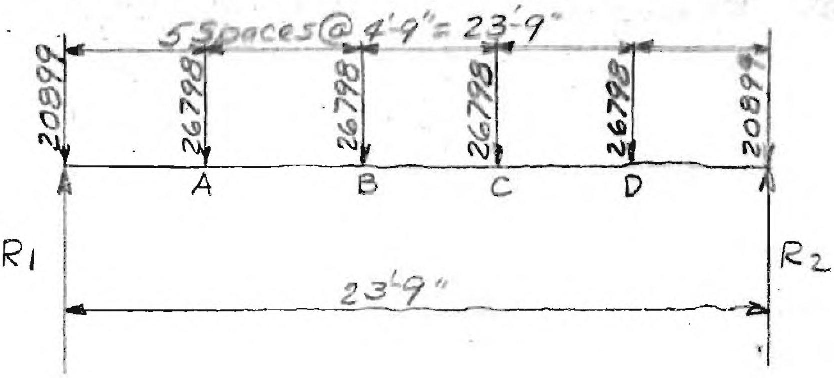
<u>Trunnion Girder</u>
See Sketch on next sheet
Dead loads of deck

$17 \times 4.75 \times 11$	=	888
$17 \times 4.75 \times 100$	=	8075
17×55	=	935
		9898
Unif. L.L. $.5 \times 4.75 \times 80$	=	1900
		11798
Concentrated L.L.		15000
		26798

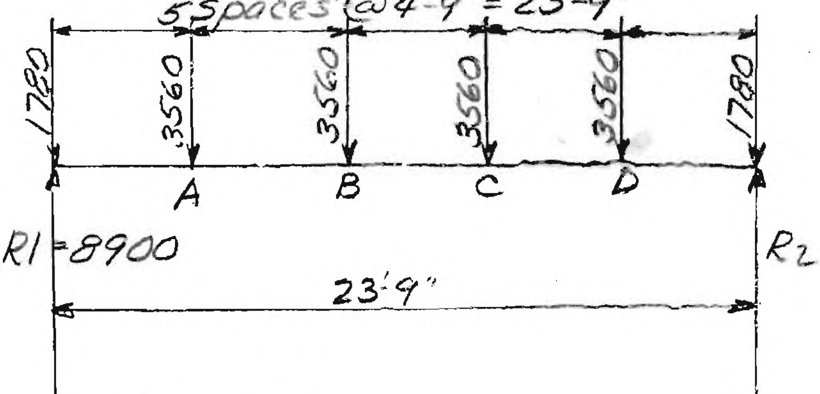
Reaction for side stringers

$= 11798 \div 2$	=	5899
+	=	15000
		20899

$V_a = 74495 - 20899 = 53596$
$V_B = 53596 - 26798 = 26798$
$R_1 = R_2 = 20899 + 26798 + 26798 = 74495$



Moms. A = 74495×4.75×12 = 4,246,215 in #
" B = (74495×9.5 − 26798×4.75)12 = 6,964,994 in #



Assumed wt. of girder

2 webs	2×7×25×15.3	=	5355
4 Cov. Pls. 5/8"	4×3×25×25.5	=	7650
8 Ls 4×4×5/8"	= 8×15.7×25	=	3140
32 Ls 3×3×3/8" Stiffeners		=	1632
			17777

17777 ÷ 23.75 = 750
4.75 × 750 = 3560

Moms. A. = 8900×4.75×12 = 508000 in #
" B = 8900×9.50 − 3560×4.75 = 676000
V_A = 8900 − 3560 = 5340
V_B = 5340 − 3560 = 1780

	L.L. + D.L.	Impact
M_A	4,754,000	
M_B	7,641,000	

Trunnion Girder Ctd. Sheet # B16

Total Shears

At support = 83395 + 25018 = 108400
V_A = 58936 + 17680 = 76616
V_B = 28578 + 8573 = 37151

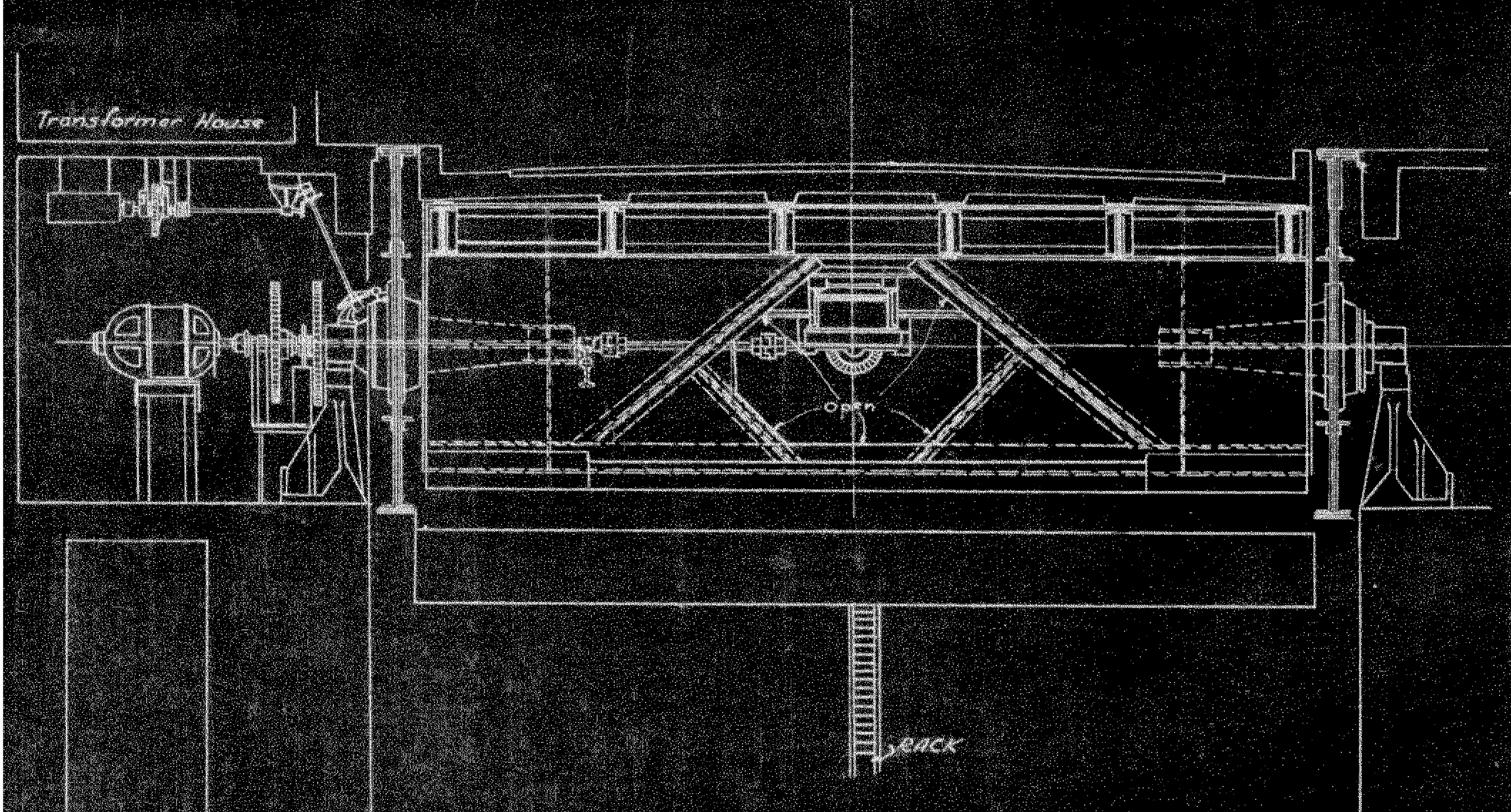
Web Req'd. 108400 ÷ 10000 = 10.84 □" Gr.
Web Used 78 × 3/8 - = 29.3 □" Gr. = 27.3 □" net
Eff. Depth = 6'6" − 3" = 6'3" = 6.25'
Flange Stress = 9,933,000 ÷ 7.25 = 1,365,000 # at B
" " = 6,180,000 ÷ = 990000 # at A
Flange Req'd. = 1,365,000 ÷ 16000 = 85.5 □" Net
Flange Used 2 - 7/8" PLs = 63.00
4 Ls 4×4×3/4" = 21.76
1/8 Web = 29.3 ÷ 8 = 3.67
87.43 Gr. = 85.43 □" net

Spacing of flange rivets
End to pt. A $\frac{10830 \times 75}{108400}$ = 7½" Both. fl. } Space Rivets 6"
Pt. A to Centre $\frac{10830 \times 75}{76616}$ = 10.6" " "

Place 8 stiffeners under each stringer
Stiffeners at end
$\frac{108400}{14000}$ = 7.7 □" use 2-3×3×3/8" with diaphrom
Stiffeners at A
$\frac{30358}{14000}$ = 2.17 □" use 2-3×3×3/8"

See Drwg. on next sheet

End. Diaphragms Max. load 350000 #
Bearing area req'd. = 21.9 □"
Use PL 18" × 1½"
Intermediate Diaphagms Max. load 250000 #
Bearing area req'd. = 15.65 □"
Use PL 10 × 1¾"
For further details see drwgs.



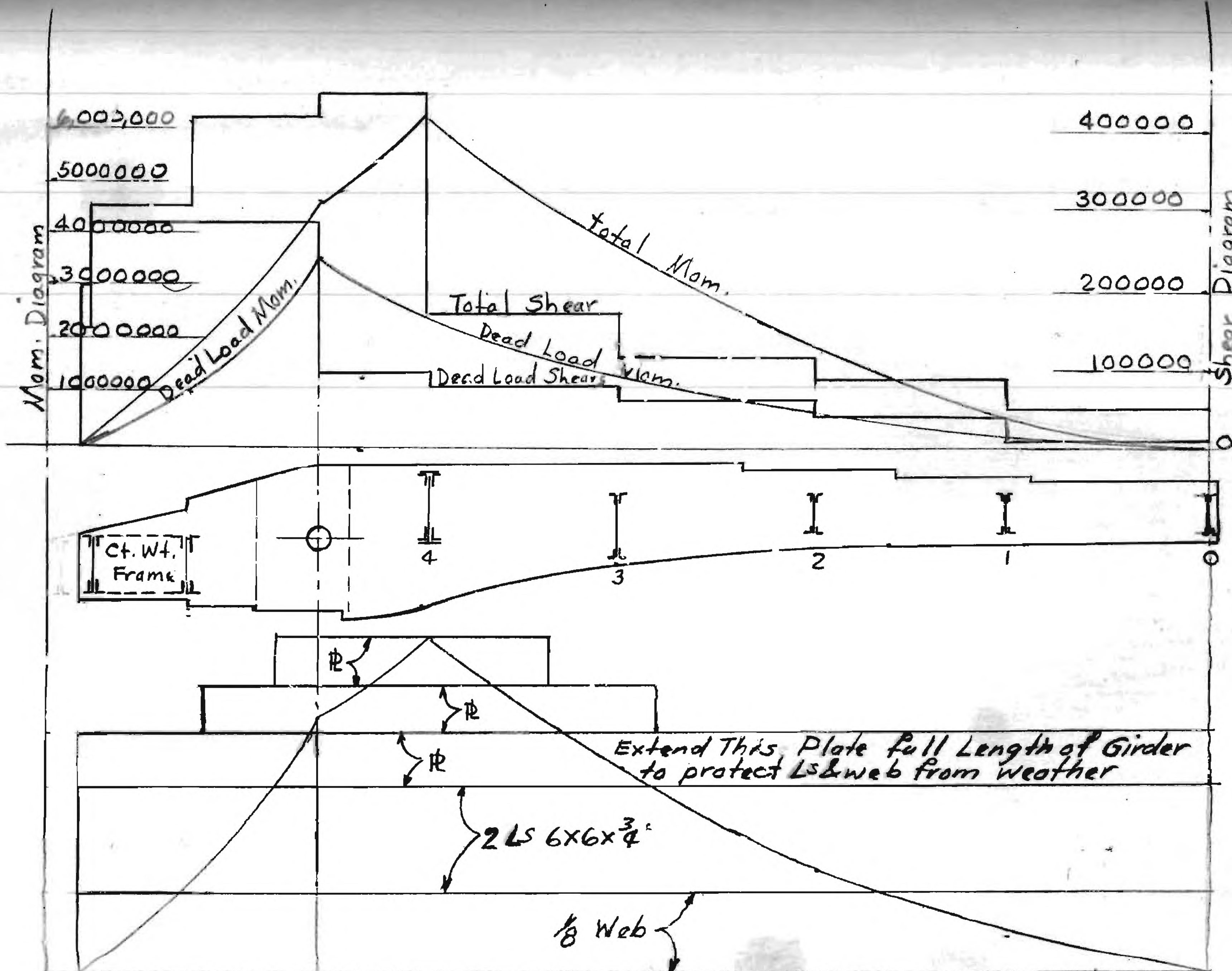
CROSS SECTION IN FRONT OF
TRUNNION GIRDER

LONGITUDINAL GIRDERS

PANEL POINT	REAR CT. WT FRAME	FRONT CT. WT. FRAME	℄ TRUNNION	(4) L. L. BEARING	3	2	1	0
Shear								
D. L.	135000	280000	280000	97600	79000	56600	34100	11600
L. L.	36000	17000	68000	280000	61500	45750	30000	20000
I	36000	17000	68000	75520	28100	20470	12820	6320
Total	207000	314000	416000	453120	168600	122820	76920	37920
Shear in Flgs.								
" " Web	207000	314000	416000	453120	168600	122820	76920	37920
Sect. Req'd. 10000# Gr.	20.7□" Gr.	31.4□" Gr.	41.6□" Gr.	45.31□" Gr.	16.86□" Gr.	12.28□" Gr.	7.69□" Gr.	3.8□" Gr.
Web used	65×9/16=36.6□	95×9/16=53.4	119×9/16=67.0□	114×9/16=64.2□	80×1/2 = 40□"	60×3/8" = 22.5□	48×3/8 = 18□	43½×3/8 = 16.3□
Moment								
D. L.		1,120,000	3,650,000	2,600,000	1,550,000	694000	172000	0
L. L.		297,000	612,000	2,752,000	1,732,375	926,500	353000	0
I		297,000	612,000	1,050,000	656,475	324,000	105000	0
Total		1,714,000	4,874,000	6,402,000	3,938,850	1,944,500	630,000	0
Eff. Depth		7.7'	9.9'	9.5'	6.6	4.9	3.8	3.7
Top Flg. Stress		233260	483000	674000	596000	598000	166000	
Bott. " "		223000	483000	684000	604940	601588	166000	

			Gr.	Net	Gr.	Net	Gr.	Net	Gr.	Net	Gr.	Net	Gr.	Net	Gr.	Net.
Flange Req'd.			14.55	14.55		30.2		42.1		37.3		37.3		10.4		
Section Used																
⅜ Web					8.38	8.38	8.02	8.02	5.00	5.00	2.81	2.81	2.25	2.25	2.03	2.03
2 Ls 6×6×¾"			16.88	13.88	16.88	13.88	16.88	13.88	16.88	13.88	16.88	13.88	16.88	13.88	16.88	13.88
1 PL			6.50	5.50	6.50	5.50	6.50	5.50	6.50	5.50	6.50	5.50	6.50	5.50	6.50	5.50
1 PL					6.50	5.50	6.50	5.50	6.50	5.50	6.50	5.50				
1 PL					6.50	5.50	6.50	5.50	6.50	5.50	6.50	5.50				
Total			23.38	19.38	44.76	38.76	44.40	38.40	41.38	35.38	39.19	33.19	25.63	21.63	25.41	21.4

Sheet # B17

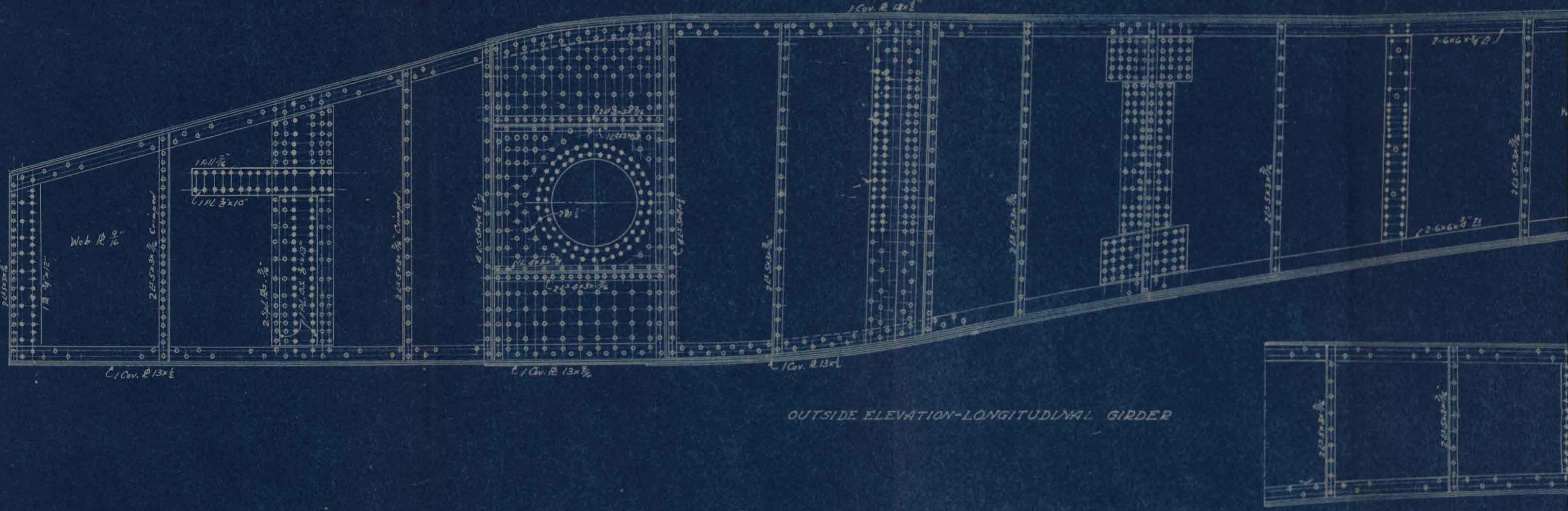


Sheet # B18

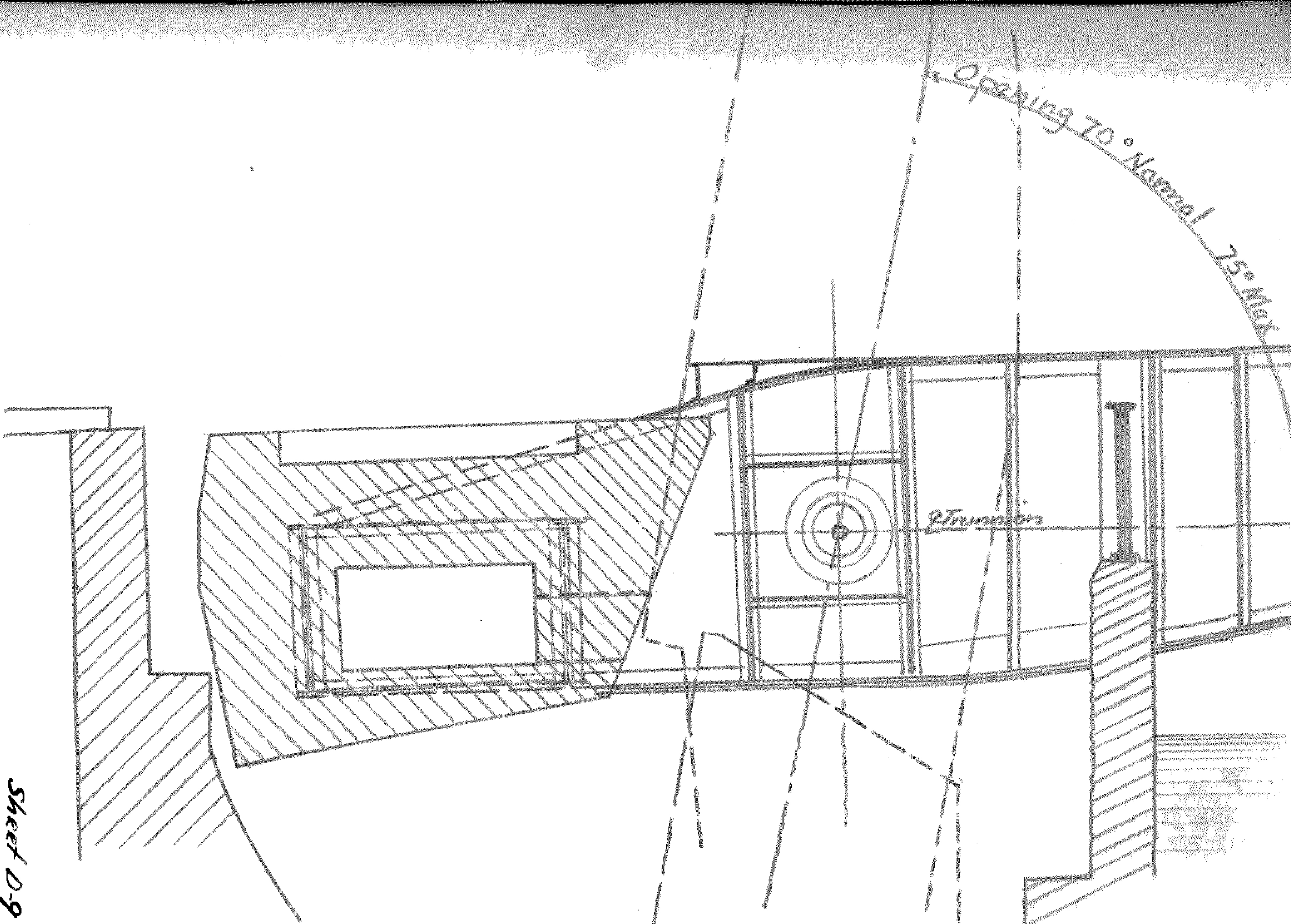
MAIN LONGITUDINAL GIRDER

OUTSIDE ELEVATION-LONGITUDINAL GIRDER

Sheet # D.1



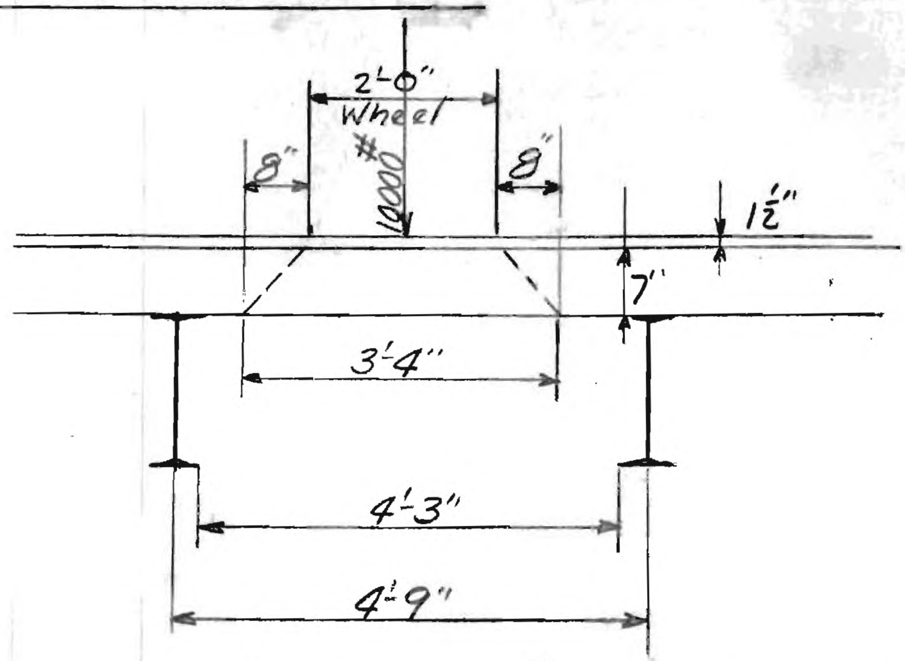
OUTSIDE ELEVATION-LONGITUDINAL GIRDER



LONGITUDINAL SECTION AT MAIN BASCULE GIRDER

Sheet D-9

FIXED PORTION



Concentrated load $10000^{\#}$ = one rear wheel
Span c-c = 4.75'
Clear span = 4.25'
Dead Loads
Asphalt Board = $11^{\#}$
Slab = $100^{\#}$
Total $111^{\#}/\square'$

D.L. Mom. $= \frac{111 \times 4.75^2}{12} \times 1.5 = 3770$

$\frac{10000}{3.33 \times 3.33} = 900^{\#}/\square'$

L.L. Mom. $= \frac{900 \times 4.75^2}{12} \times 1.5 = 30400$
Impact @ 30% $= 3040$
Total Mom. $= 37210$

$bd^2 = \frac{M}{K}$, $d = \sqrt{\frac{37210}{12 \times 126}} = \sqrt{24.6} = 4.96''$
Make 'd' = 7"

Slab D.L. $4.75 \times 111 = 530^{\#}$/ft. of stringer
Stringer 50
580

Referring to Sect. "B" on next sheet
D.L. Mom. $= 580 \times 11.25 \times 9.75 - \frac{580 \times 9.75^2}{2} = 35900$ ft
L.L. Mom. $= 8710 \times 12.75 - 5000 \times 9.5 = 63500''$
Impact $= 19100''$
Total Mom. $118500''$

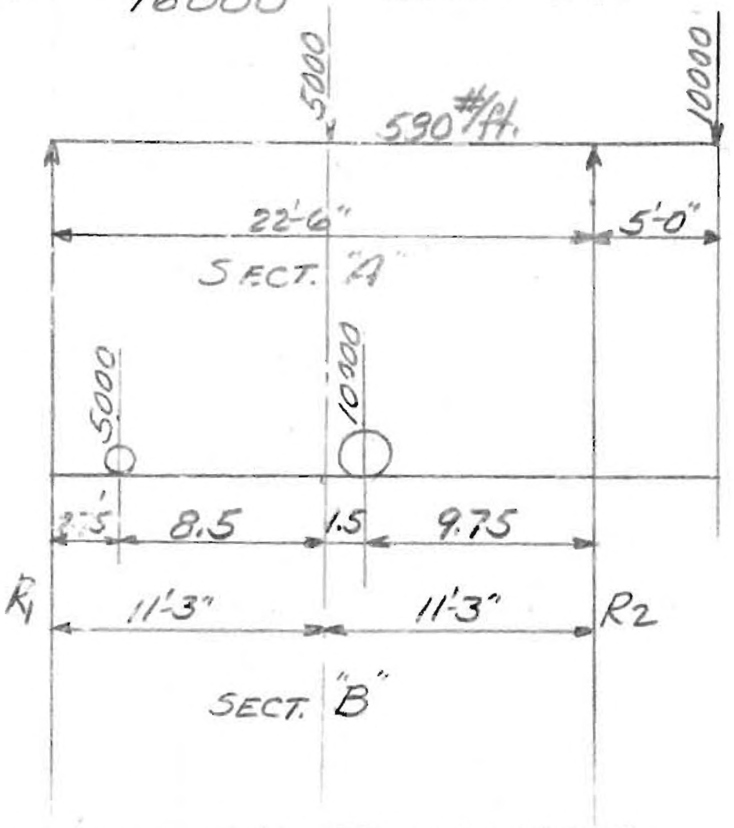
	Moment	Shear
D.L.	35900	7530
L.L.	63500	13000
I	19100	3900
Total	118500 ft.#	24430#

Sect. Mod = $\frac{118500 \times 12}{16000}$ = 88.8

Use 18" I @ 60#

Overhang 10000×.5	=50000 ft#. L.L.Mo
D.L. 580×.5×2.5	= 7250
Impact	=50000
	107250

S.M = $\frac{107250 \times 12}{16000}$ = 80.40 O.K



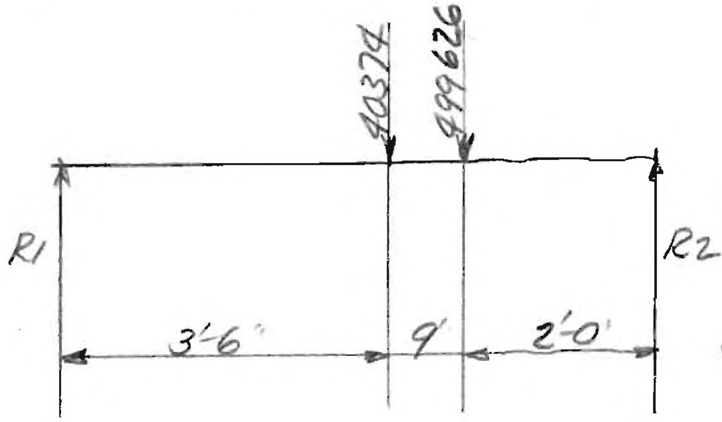
DESIGN OF TRUNNION

The load on main trunnions equals the resultant of

(1) The dead load of entire bascule leaf
(2) The maximum wind load.
(3) The pinion reaction

Trunnion Girder Reaction	=108400
Trunnion Wt. 7×.7854×T²×490	= 2750
	111150

DESIGN OF TRUNNION CTD.

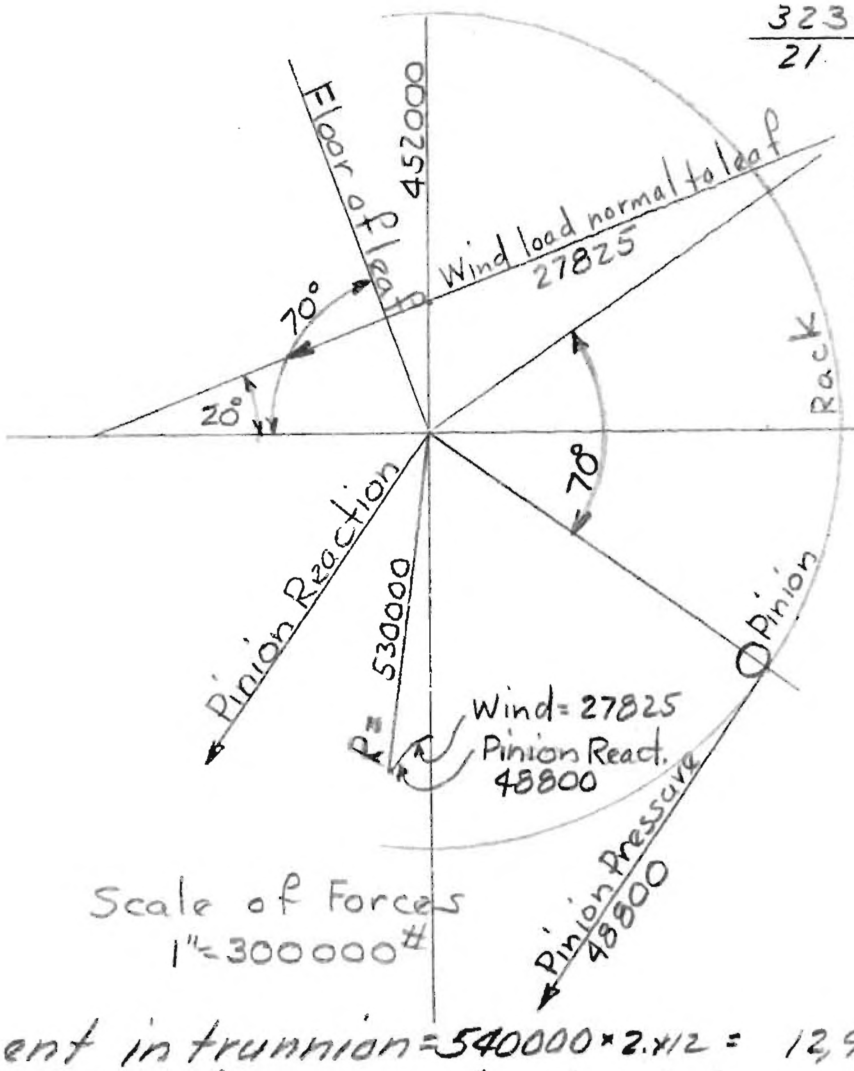


Movable D.L. = 377000
Impact @ 20% = 75400
452400

Bearing area @ 1400 #/□"
$= \frac{452400}{1400} = 323$ □"

$\frac{323}{21} = 15.35"$

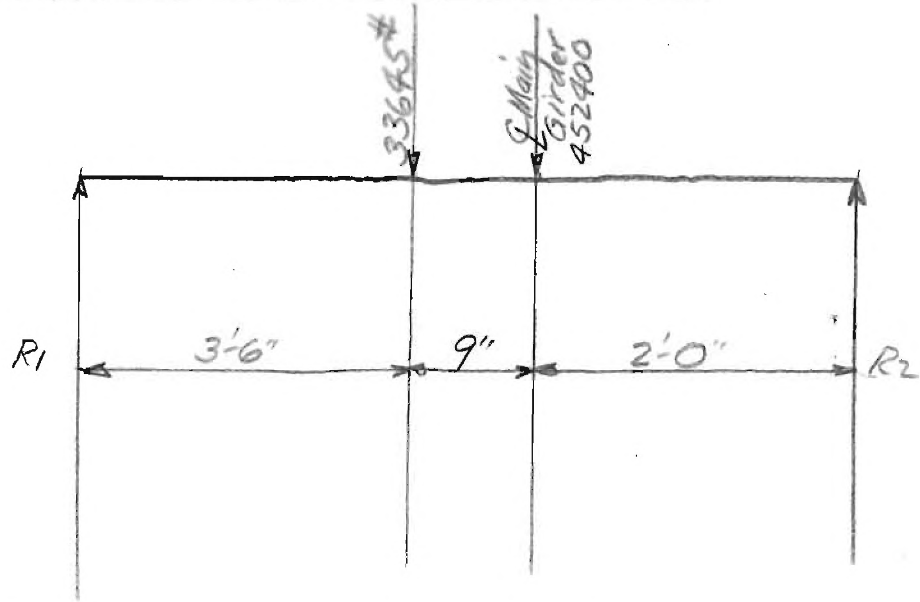
∴ Width of bearing = $15\frac{3}{8}"$
Dia. of each trunnion at ℄ of main girder = 21"
phosphor bronze bushings



Moment in trunnion = 540000 × 2.×12 = 12,960,000 in.#
Taking dia. of trunnion at main girder = 21" & counterbore = 3.3" Then $f_s = \frac{Mc}{I}$, $c = \frac{d}{2}$, & $I = \frac{\pi(d^4 - d_1^4)}{64}$

$$f_s = \frac{32(12{,}960{,}000)}{\pi\left(\frac{21^4 - 3.3^4}{11}\right)} = 7460 \text{ \#/□"}$$ which is safe.

DESIGN OF TRUNNION CTD.



Movable Dead Loads = 280000 + 18000 + 23000 + 22500×2 + 11700 = = 377000

D.L. + L.L from fixed span & trunnion girder = 8900 + 2(9898) + 4949 = 33645

Total = 410645

Impact @ 20% = 82129

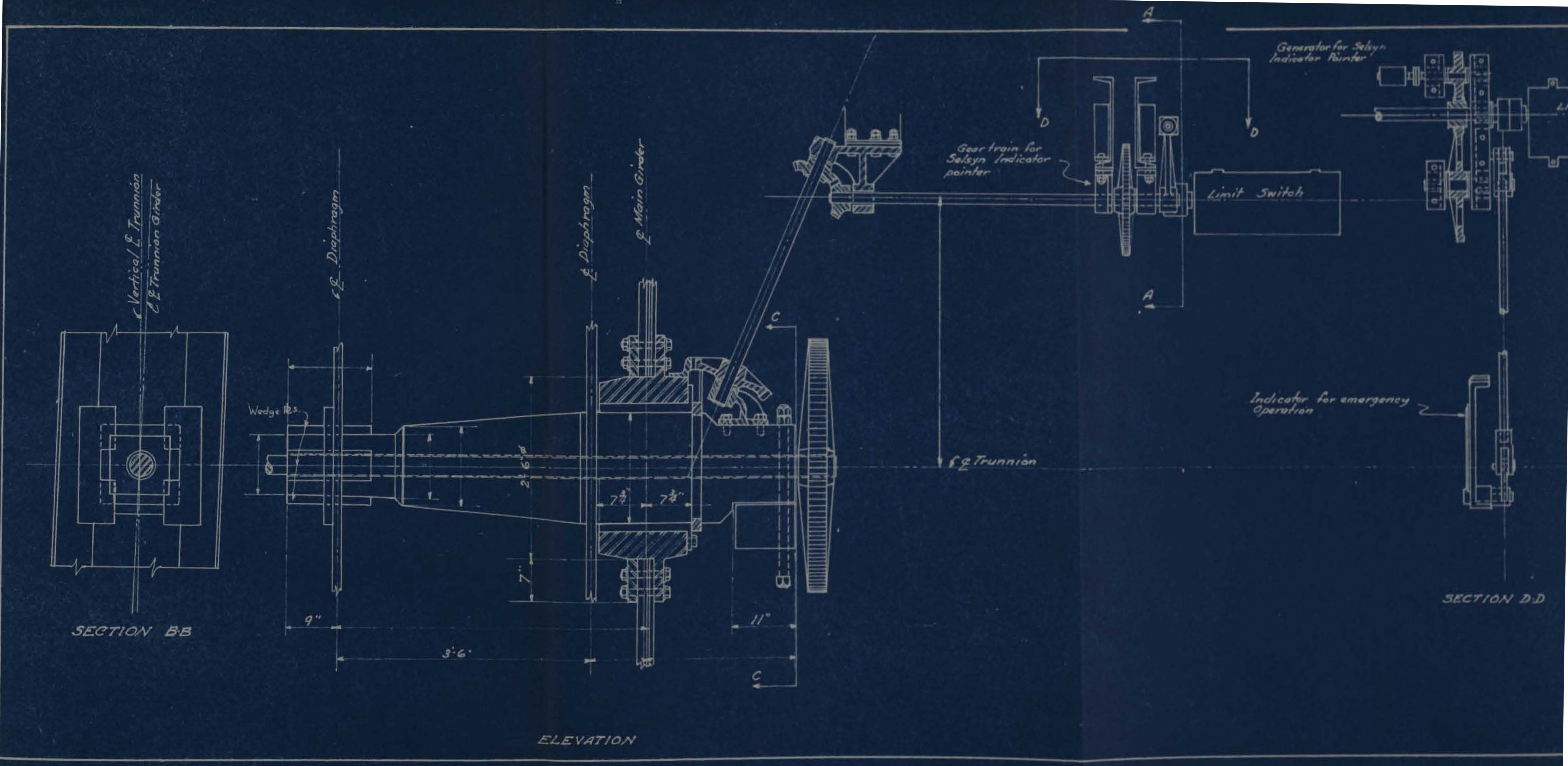
492774

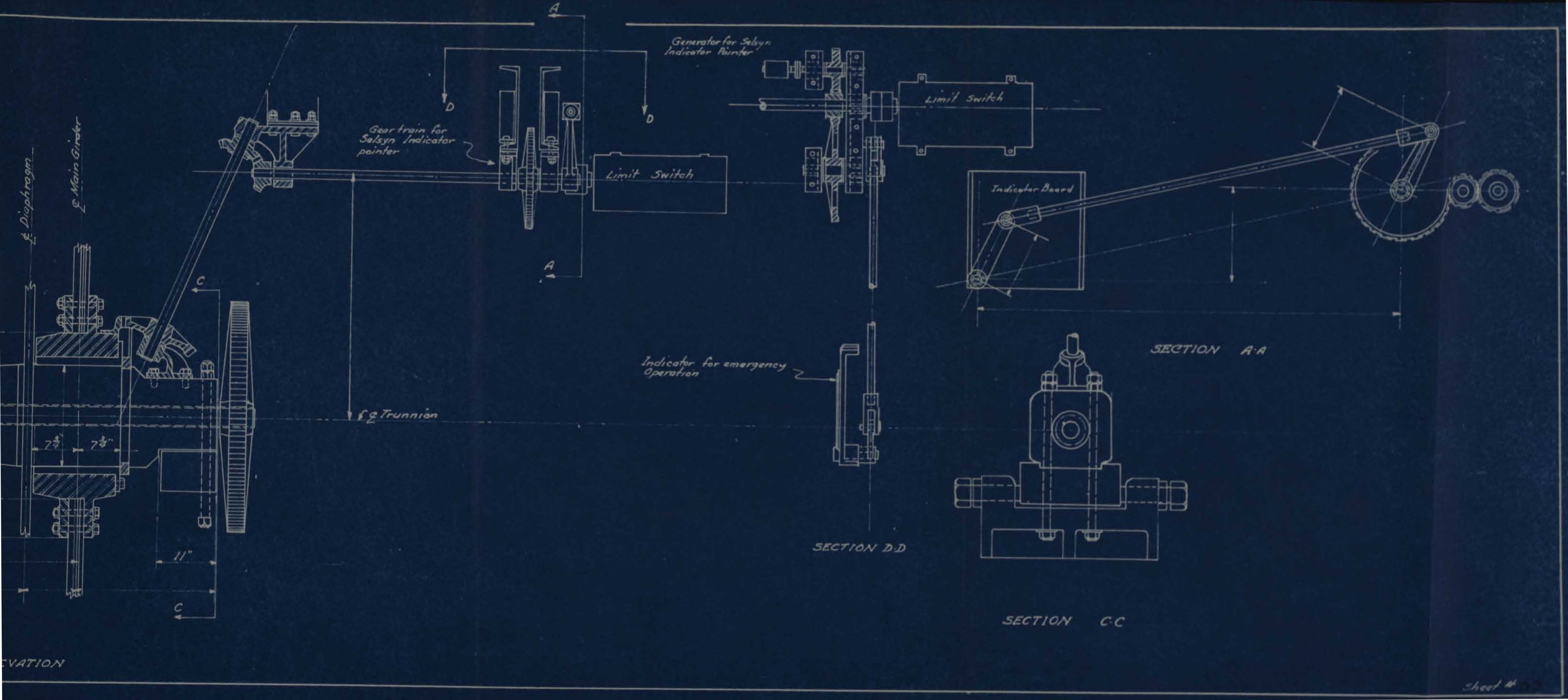
Wind = 1855 × 15 = 27825

Trunnion Shoe Reaction = 540000#

$\frac{540000}{110.2}$ = 4900#/□"

Steel on Cast Steel


Vertical ℄ Trunnion
℄ Trunnion Girder
℄ Diaphragm
℄ Diaphragm
℄ Main Girder
Wedge Pls.
2'-6"
7"
9"
3'-6"
7 3/4"
7 3/4"
11"
℄ Trunnion
Gear train for Selsyn Indicator pointer
Limit Switch
Generator for Selsyn Indicator Pointer
Indicator for emergency Operation
A
A
C
C
D
D
SECTION B-B
ELEVATION
SECTION D-D



A
A
D
D
Generator for Selsyn Indicator Pointer
Gear train for Selsyn Indicator pointer
Limit Switch
Limit Switch
Indicator Board
℄ Diaphragm
℄ Main Girder
C
C
℄ Trunnion
7¾"
7¾"
11"
Indicator for emergency Operation
SECTION A-A
SECTION D-D
SECTION C-C
EVATION
Sheet #

See Sketch on last sheet

33645 + 20% impact = 40374

540000 − 40374 = 499626

$R_1 = \dfrac{-40374 \cdot 2.75 - 499626 \cdot 2}{6.25} = 178000\#$

Shear on Diaphragm trunnion girder due to trunnion react.

= 178000 + 2(26779 + 3600) = 239000#

Bearing area req'd. $= \dfrac{239000}{16000} = 15^{\square''}$

Shaft S2 (See gearing plans)

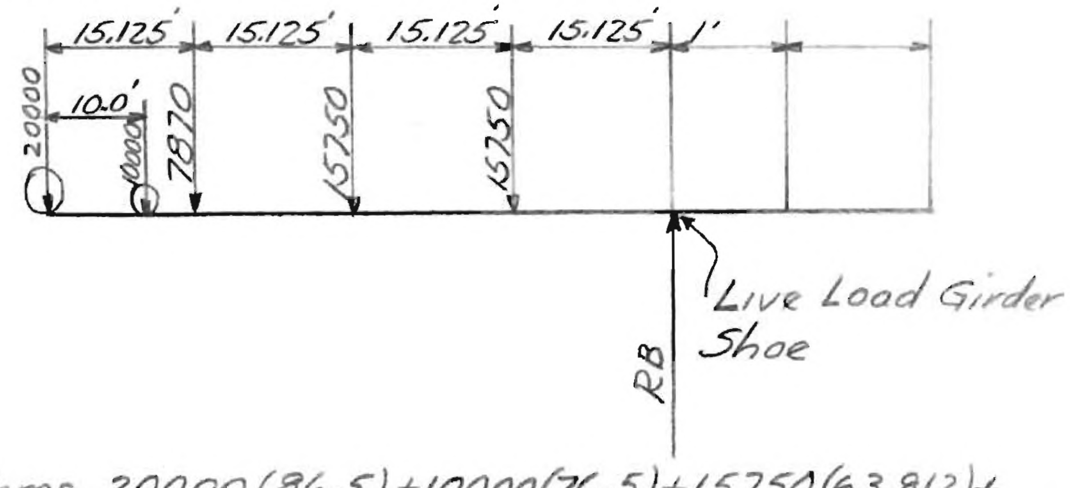
torque = 30400 in#

shearing stress due to torque $= f_s = \dfrac{M_c}{I_p}$

I_p = polar moment of inertia $= \dfrac{\pi d^4}{32} = \dfrac{16 M_t}{\pi d^3}$

$d = \sqrt{\dfrac{16 \cdot 30400 \cdot \underbrace{10}_{\text{factor of safety}}}{3.1416 \times 50000}} = \sqrt{31} = 3.13''$ Say $3\frac{1}{4}''$ dia.

LIVE LOAD GIRDER



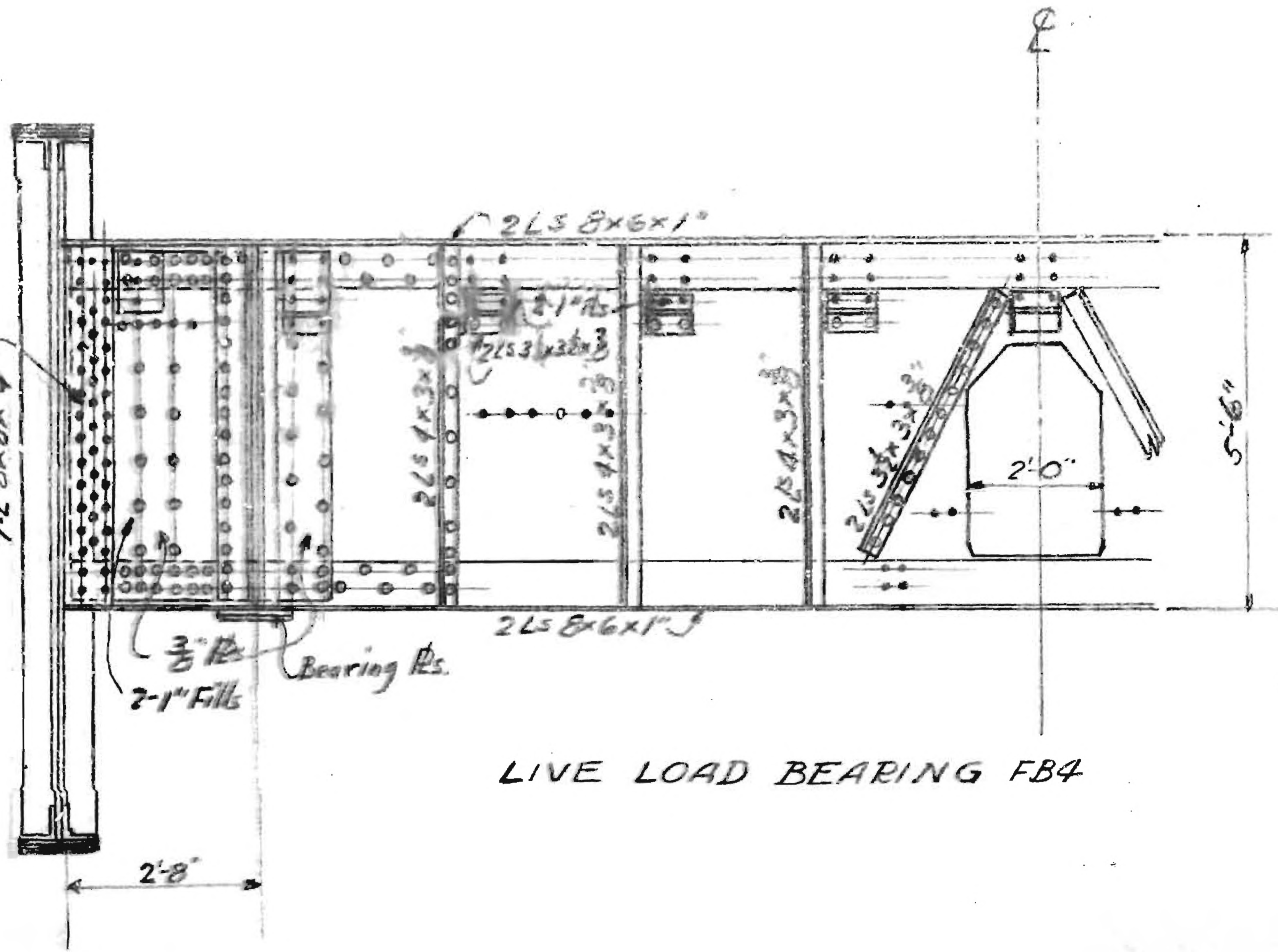
L.L. Moms. 20000(86.5) + 10000(76.5) + 15750(63.812) + 15750(48.687) + 15750(33.562) + 21000(26) = 26(R_B)

R_B = 472000#

D.L. Moms. 11700(87) + 22500(71.875) + 22500(56.75) + 23000(41) + 18000(26) + 280000(3.88) = 26 R_B R_B = 247000#

Dead Loads on L.L. Girder

$1\frac{1}{2}''$ Asphalt	$12 \times \frac{85}{12}$	= 85
Flooring	12×20	= 240
Stringers	$12 \times 45 \times \frac{1}{2.42}$	= 250
Total		799#/ft.
Say		800 "



LIVE LOAD BEARING FB4

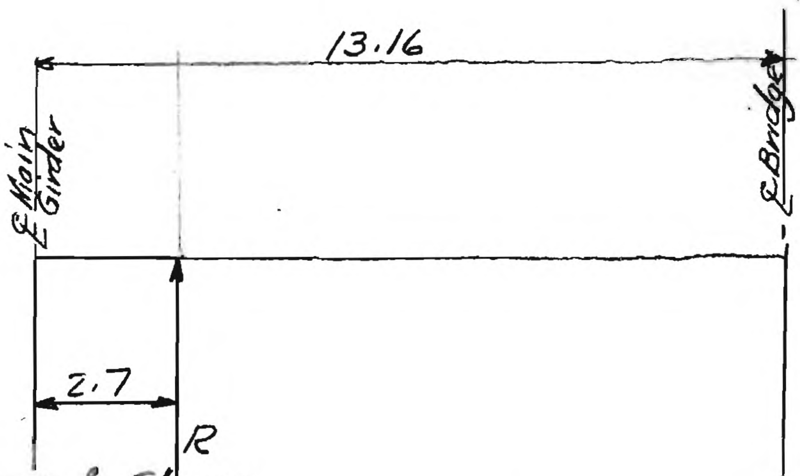
Sheet # 11-D

LIVE LOAD GIRDER CTD.

800 (13.16)	=	10550
800 (2.5)	=	2000

L.L on L.L. Girder

12×80	=	960
960(12)	=	11500
960(2.5)	=	2400



D.L. Moms. & Shears

Mom. at center = -1000(13)+ 10000(10) - 2000(9.7)-2000(7.3)- 2000(4.8) - 2000(2.4) , R = = +38600

Shear (Left of support) 10000-1000 = 9000

Shear (Right " ") = 9000

L.L. Moms. & Shears

Mom. at centre -1200 (13)+12000(10)- 2400(9.7)- 2400(7.3) - 2400(4.8)- 2400(2.4) R = +46400

Shear left of L.L. Bearing =12000-1200 = 10800

472000 - 12000 = 460000

L.L. shear at left of L.L. sup't. = 460000

L.L. " " Rt. " L.L. " = 460000-472000+10800 = 22800

D.L. Mom. left of L.L. Supt. = -237000(2.7) = -638000

D.L. " at centre of bridge = -237000(13.16)+ 247000(10.46) = -540000

L.L. Mom. left of L.L. supt. = - 460000× 2.7. = -1,240,000

L.L. " at centre of bridge = - 460000(13.16)+472000(10.46) = -1,120,000

LIVE LOAD GIRDER					
	Moments			Shears	
	At L.L. Bearing	At ℄ Bridge		outside L.L. Bearing	inside L.L. Bearing
D. L.	638000	540000	D. L.	9000	9000
L. L.	1240000	1120000	L. L.	460000	22800
I	620000	560000	I	230000	11400
Total	2498000	2220000	Total	699000	43200
Eff. Depth	5.00'	5.00	Web Reqd. at 10000	69.9□" net	43.2□" net
Flg. Stress	500000	444000	Use 1 Pl 66×½	33.0□"	33.0□"
S.R. @16000	31.3□" net	31.2□" net	2 Pls. 58×⅜	43.5	43.5
Sect. Used	Gr.	Net	Total	76.5□"	76.5□"
⅛ Web	5.44	5.44			
2 Ls 8×6×1"	26.00	25.00			
Total	31.44	30.44			

Counterweight Frames

Rear Frame closed & L.L. on

$\frac{510000}{26.33} = 19400$ #/ft.

$19400 \div 2 = 9700$ #/ft. of frame

When movable leaf is closed the downward reaction due to anchorage must be added to the above, this equals (at rear frame)

$36000 \div 13.16 = 2730$ #/ft. of frame.

At front Ctwt. frame $16800 \div 13.16 = 1275$ #/ft. of fram

	Rear Ctwt. Frame	Front Ctwt. Frame
Dead Load	= 9700	9700
Anchorage React.	= 2730	1275
I @ 30% of D.L.	= 2910	2910
Total in #/ft. of Frame	= 15340	13885

Max. Bend. Mom. $= \frac{15340 \times \overline{26}^2}{8} = 1,300,000$ ft.# Rear Frame

" " " $= \frac{13885 \times \overline{26}^2}{8} = 1,170,000$ ft.# Front Frame

Max. Shear 15340×13.16 = 202 000# @ Rear Frame

Max Shear 13885×13.16 = 182 500# @ Front Frame

<u>Rear Frame</u> – Span 26 feet
Depth back to back of ∠s = 5'-6"
Eff. Depth = 5'-0"
Net web area req'd. = $\frac{202000}{10000}$ = 20.2□"
$\frac{20.2}{66}$ = .306 use 3/8" Web

Flange stress = $\frac{1,300000}{5.0}$ = 260000#

Req'd. flange area = $\frac{260000}{16000}$ = 16.3□" net

Flange Used	Gr.	Net
1/8 of web = (1/8).375(64)	3.00	3.00
2 ∠s 6×6×½ =	11.50	9.50
1 Cov. ℔ 13×½ =	6.50	5.50
	21.00	18.00

Note. Cover plates to be carried full length of girder, both top & bott. flanges
Stiffeners - use 2-5×3×3/8" ∠s spaced as shown on sketch.

<u>Front Counterweight Frame</u>

Depth back to back of ∠s = 5'-6"
Eff. Depth = 5'-0"
Net web area req'd. = $\frac{182500}{10000}$ = 18.25□"
18.25÷66 = .276 use 5/16" = .313
64× 5/16 = 20.0□" net
Flange stress = $\frac{1,170,000}{5.0}$ = 234000#
Req'd. flange area = 234000÷16000 = 14.65□" net

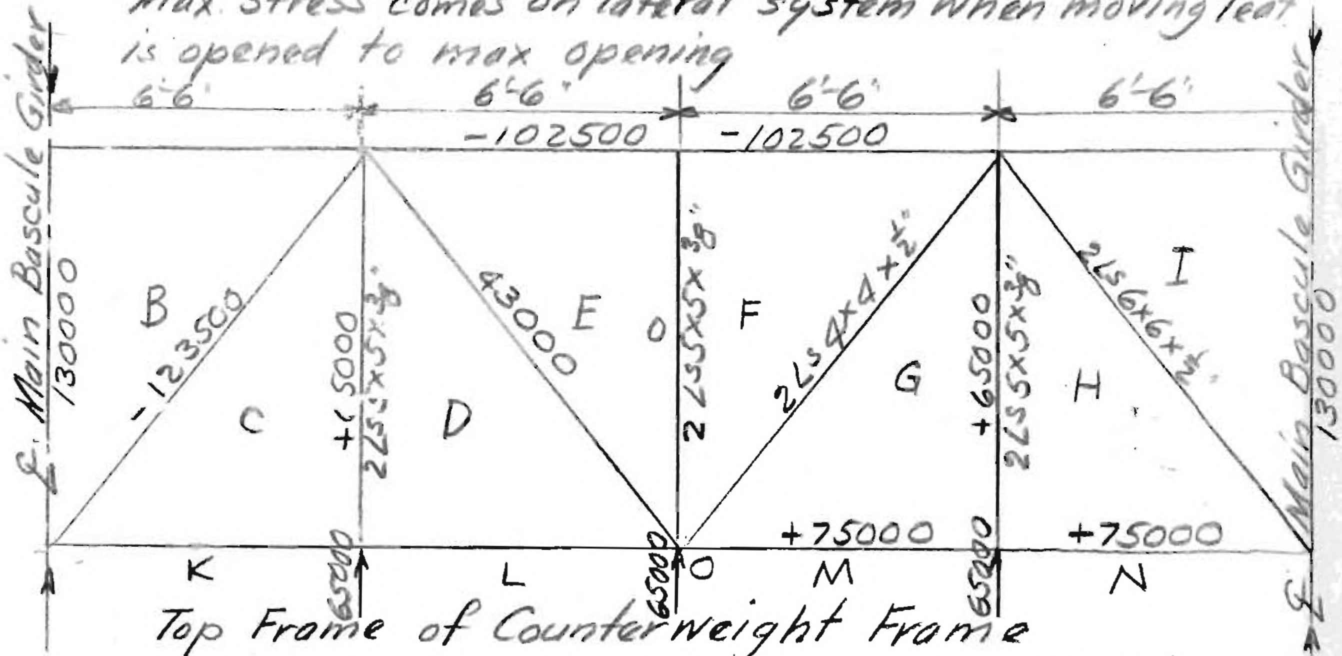
Flange Used.	Gr.	Net
1/8 web = (1/8) .375×64 =	3.00	3.00
2 ∠s 6×6×½ =	11.50	9.50
1 Cov. ℔ 13×½ =	6.50	5.50
	21.00	18.00

See sketch

Lateral System	Sheet # B27

Top lateral

Max. stress comes on lateral system when moving leaf is opened to max opening



Top Frame of Counterweight Frame

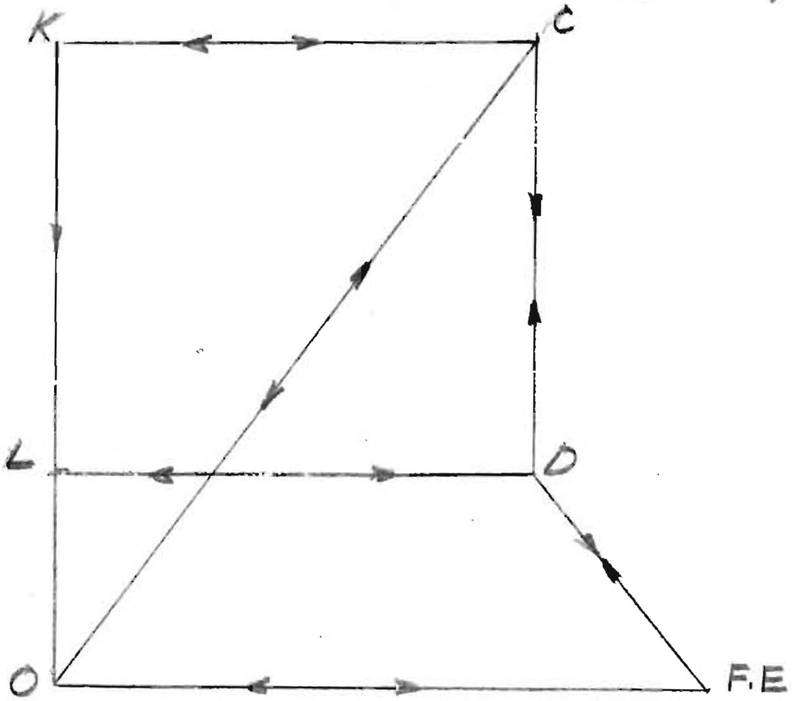
− = Tension

+ = Comp.

Load on concrete = 9700

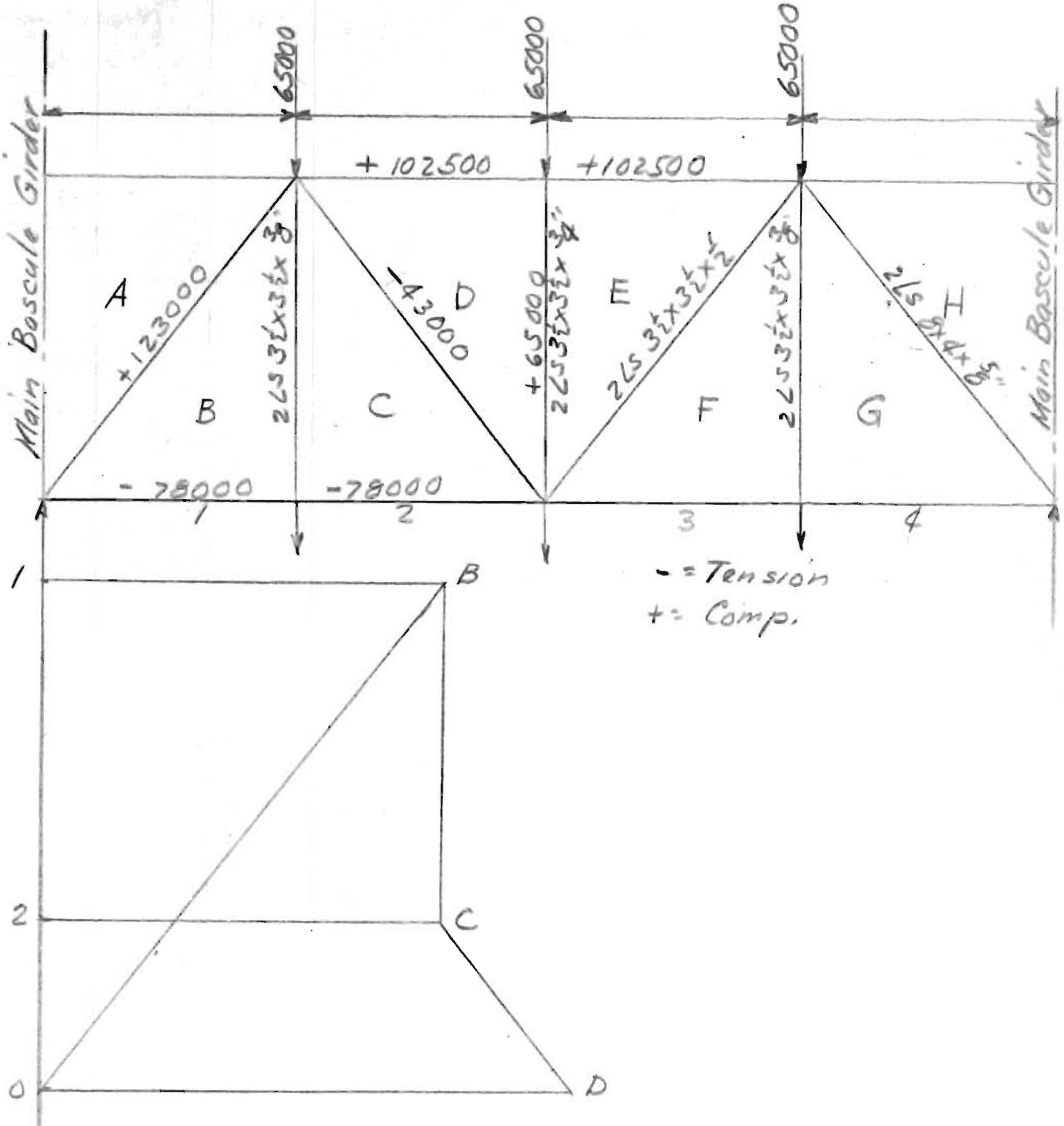
Encased steel = 300

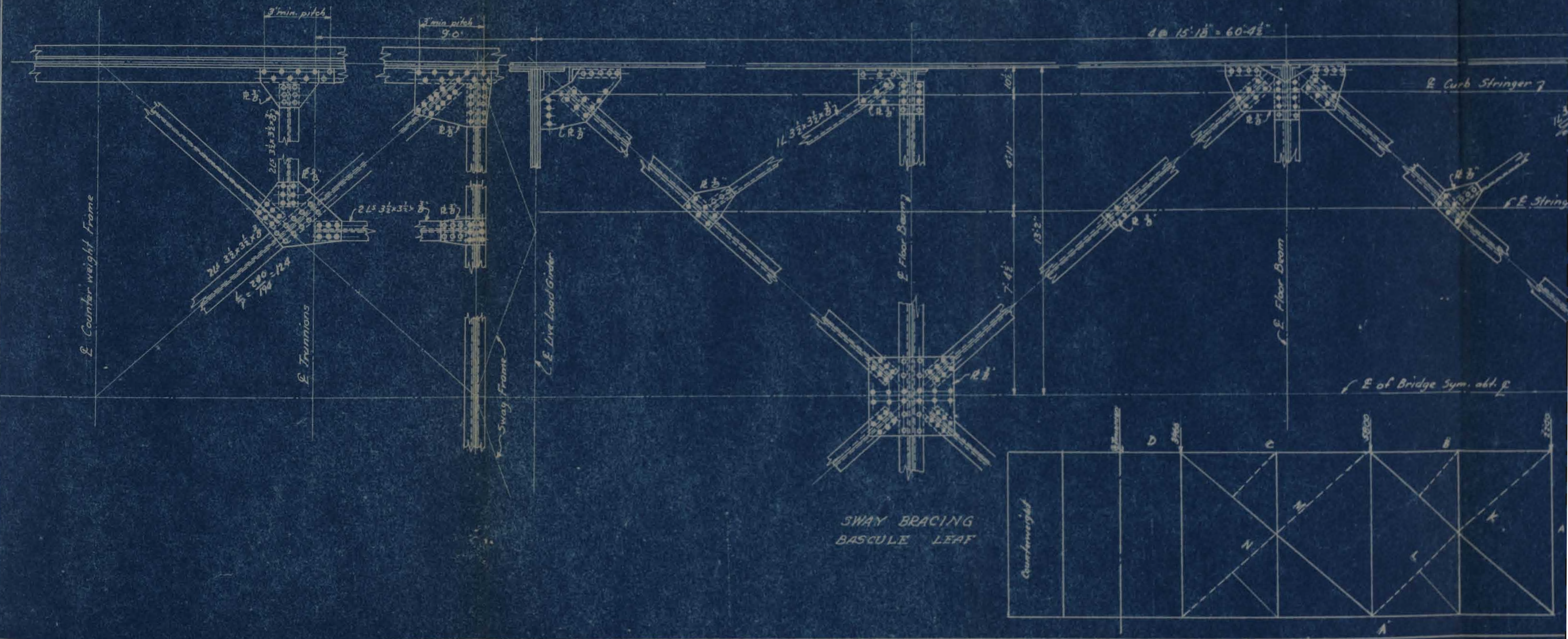
10000 #/ft. of ea. lateral frame



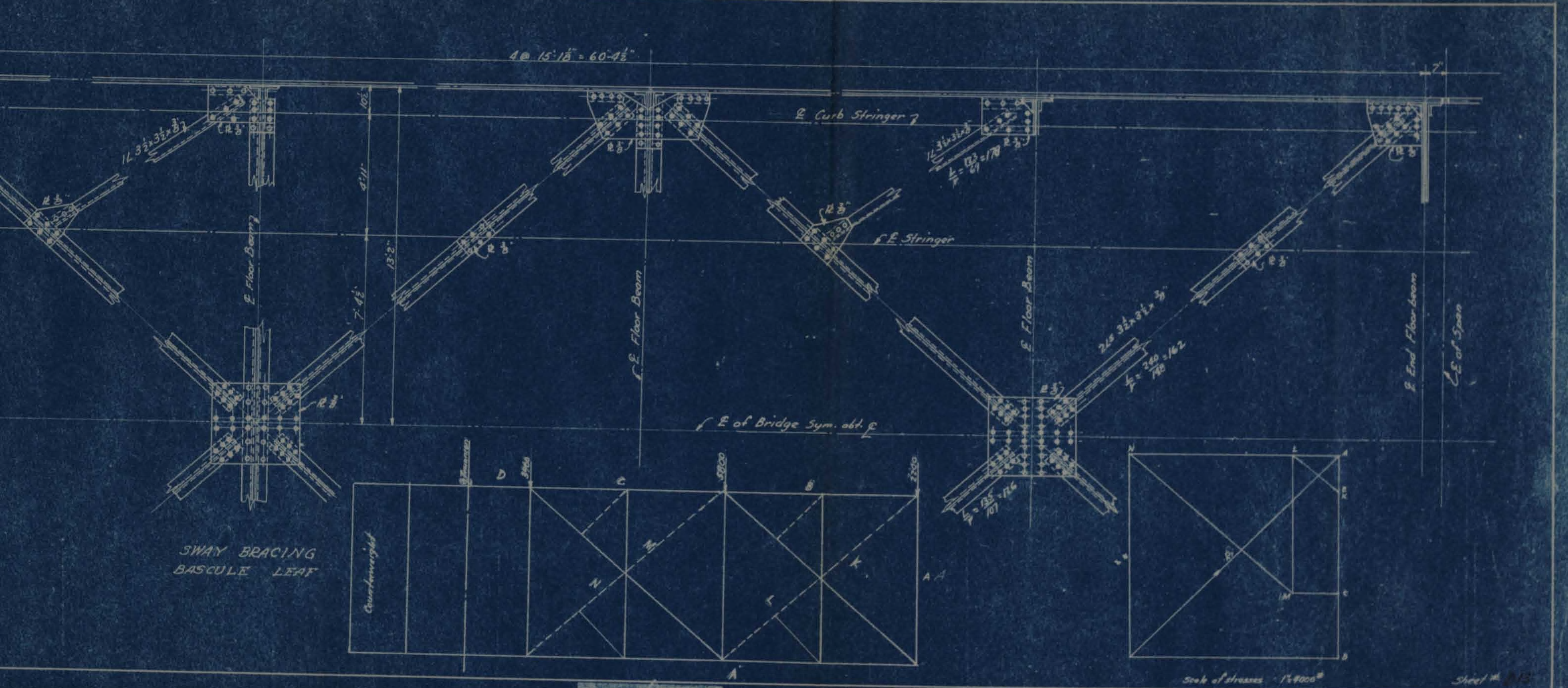
1" = 30000 #

Lateral System
Bottom lateral





SWAY BRACING
BASCULE LEAF



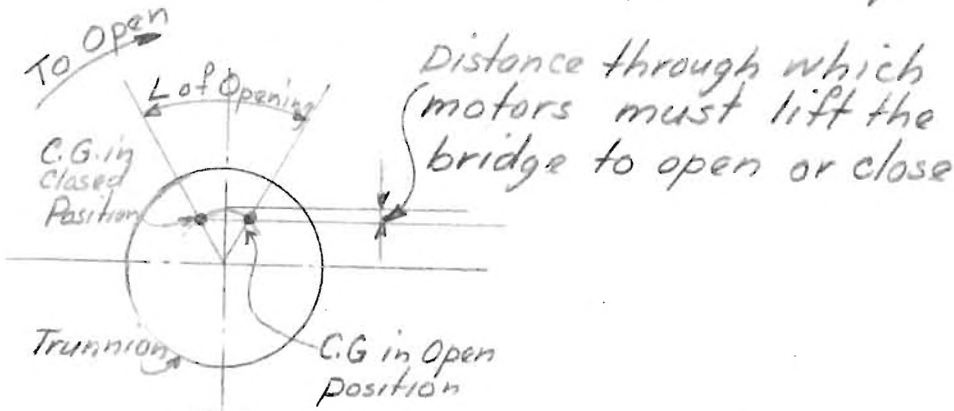
SWAY BRACING
BASCULE LEAF

Scale of stresses 1" = 4000#

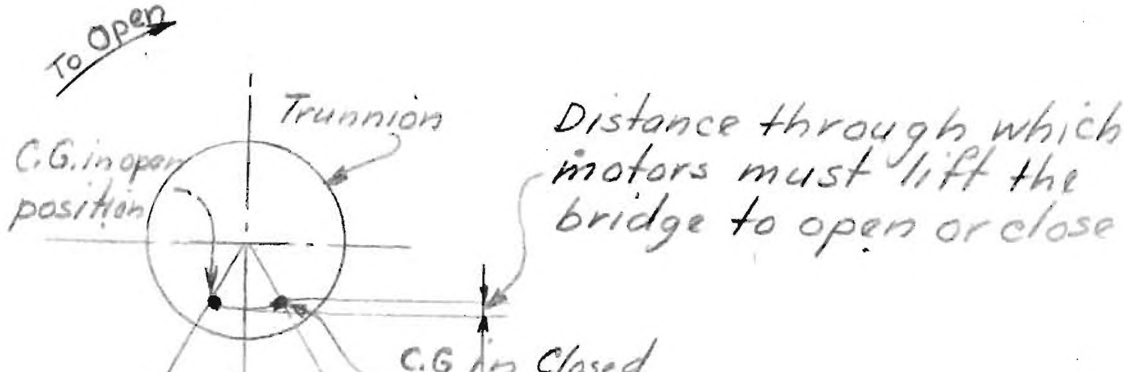
Sheet #

Locating Trunnions

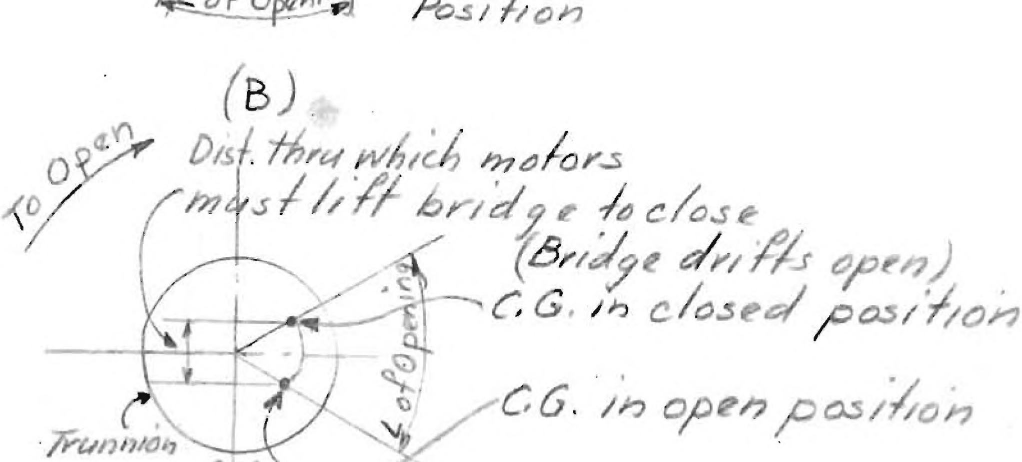
It is generally considered customary to so locate the trunnion, that a line passing through the centers of gravity of the moving leaf and the counterweight will pass through the exact center of the trunnion. This condition is difficult to obtain in practice and, too, is not desirable. It is more desirable to so locate the center of gravity of the total load on the trunnion such that the motors are working only half or part of the time of operation.



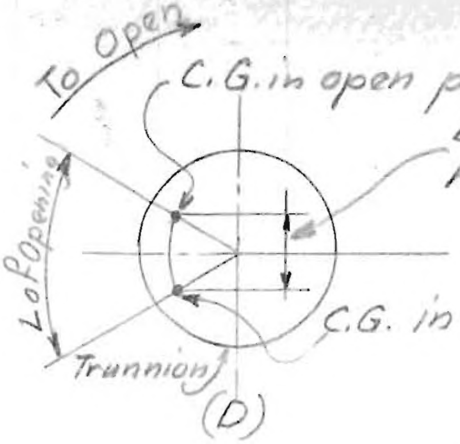
(A)



(B)



(C)



If the centre of gravity of the total load be eccentric with centre of trunnion, it will fall in one of the four quadrants as shown in sketches A-B-C-D.

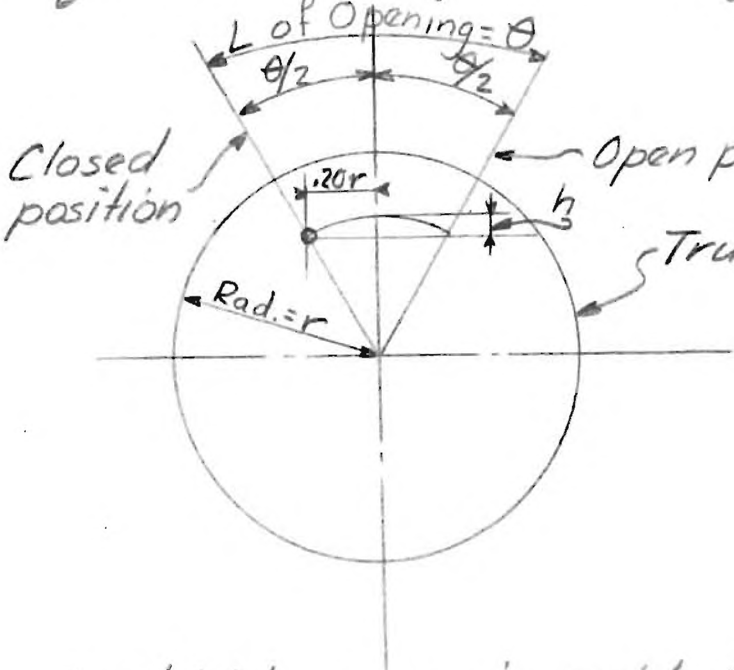
In C & D too much power will be consumed. In sketch B the bridge starts open of its own accord, and requires power on closing until the leaf is completely closed. These two conditions are unsatisfactory becau in the first case, in the event that the moving leaves become unlocked, the bridge will swing open of its own accord, in the second case, the application of power at the instant tha the live load shoes come in contact with their bearings may cause serious overstress to the machinery or bascule girder

In case A the bridge will swing closed of its own accord after it is half down, and, therefore, unless it is working agains a wind, it will require power only thru the first half of its travel. At the half point the operator will be able to shut off the motors, and devote the rest of the time entirely to the brakes in bringing the leaf to a smooth seating.

The centre of gravity of the total load should be placed at such a distance from the centre of the trunnion, that, it will cause a moment only necessary to rotate the leaf again

the friction of the bearings and machinery. this friction is usually assumed at 15 to 20 percent of the total weight of the moving leaf
This moment = Wrk
W = load on trunnions
k = Coef. of friction of the trunnions on their bearings
r = Radius of trunnions
The dist. x ahead of the geometric trunnion centre at which centre of gravity of total load is placed is such that Wx = Wrk ∴ x = rk
(assuming k = 0.20), x = .20 r

The distance necessary above the centre of the trunnion is the distance to the intersection of a vertical line thru C.G. with a line at an angle of $\frac{\theta}{2}$ with the vertical axis of the trunnion. With the C.G. in this position the amount of power required to operate will be the same in both directions, and the force holding the bridge open is equal to that required to hold the bridge fully closed.



A slight power is req'd to operate the leaf through the distance "h". (friction and unbalanced power) This is provided for in the motor design

C.G. of moving leaf.

$$\frac{11700\times70+22500\times54.875+22500\times39.75+23000\times29.125+18000\times9}{97700}=37.$$

∴ x dist. of C.G of moving leaf = 37.3' from trunn

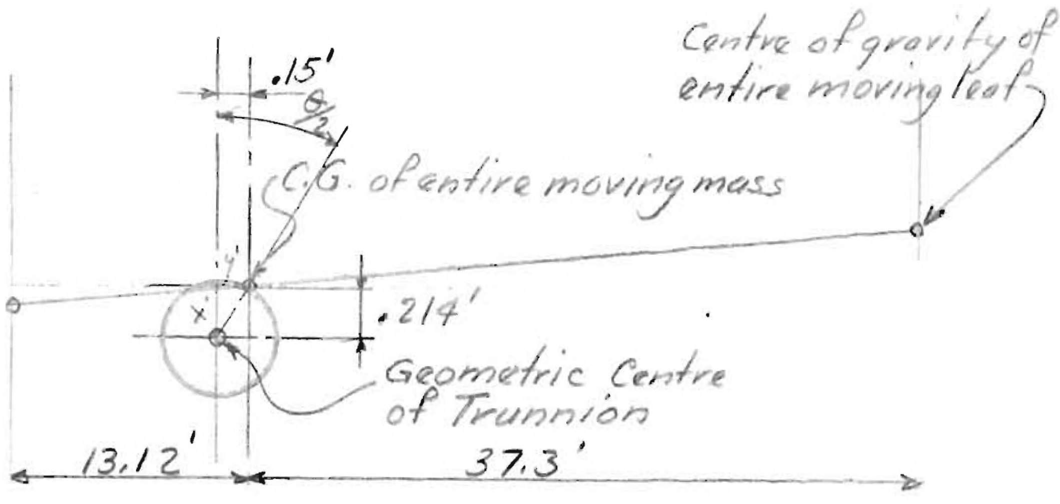
x dist of C.G of counterweight arm from trunnion = 13.12'

Determining Y dist. C.G of trunnion

104.5 ×	4.41	=	461.0
14.9 ×	2.6	=	38.8
12.3 ×	9.15	=	112.5
2.5 ×	8	=	20.0
2.0 ×	4	=	8.0
1.0 ×	1.75	=	1.75
137.2			642.05

$\frac{642.05}{137.2}$ = 4.66' from upper plane of Ct. wt.

$r = 9''$ $.20r = 1.80'' = 0.15'$



$\theta/2 = 35°$

$\tan 35° = \frac{y'}{x}$, $x = \frac{y'}{.700} = \frac{.15}{.700} = .2142'$

BASCULE PIER

In finding the maximum footing loads the following items are computed

Dead Loads

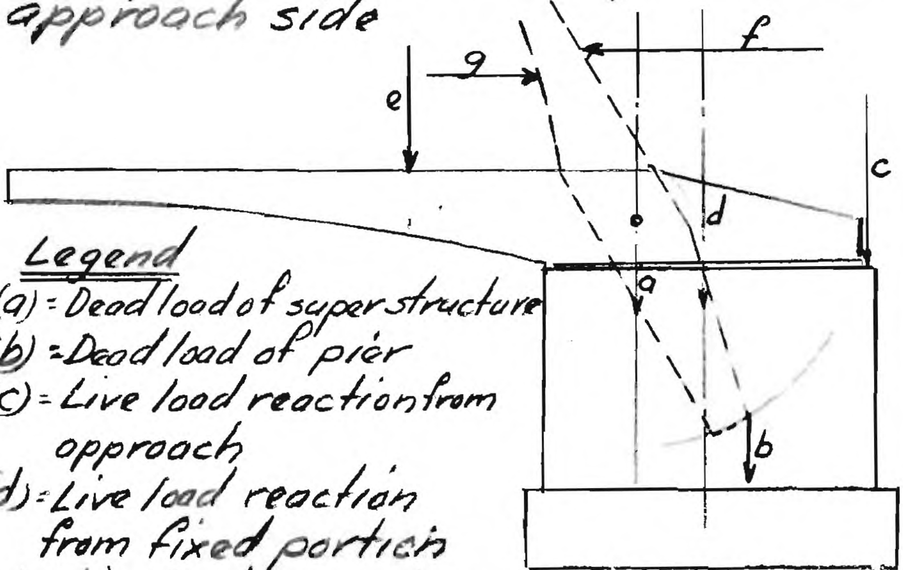
(1) Toe pressure and heel pressure, leaf down
(2) " " " " " " up

Live Load

(3) Toe pressure, full live load on approach and fixed portion, no live load on moving leaf.
(4) Toe pressure and heel pressure, full live load on moving leaf no live load on approach or fixed portion.
(5) Toe pressure and heel pressure, full live load on approaches, fixed span and moving leaf.

Wind Load (Leaf raised to full open)

(6) Toe pressure and heel pressure with wind from river side
(7) Toe pressure and heel pressure with wind from approach side



Legend

(a) = Dead load of super structure
(b) = Dead load of pier
(c) = Live load reaction from approach
(d) = Live load reaction from fixed portion
(e) = Live load on moving leaf
(f) = Wind load from approach
(g) = Wind load from river

Dead Loads of pier

Base	$38 \times 10 \times 50 \times 150 =$	2,850,000
	$34 \times 22 \times 50 \times 150 =$	5,610,000
		8,460,000
Deduct	$34 \times \frac{1}{4}(46^2 \times .7854)150 =$	2,122,875
		6,337,125

Fixed Deck

D.L.	= 10000 #	
L.L. 13×27×80	= 28080 #	} 36.504 #
I	8424 #	

Approach Span Reactions

D.L.	= 120000 #
L.L.	30100 #
I	9030 #
	159130 #

Item (1)

Taking moments about "a"

$$\frac{-120000(2.5)-6{,}337{,}125(20)-247000(35)}{6{,}704{,}125}=x$$

$$\frac{-300000-126{,}742{,}500-8{,}645{,}000}{6{,}704{,}125}=x$$

$$x=\frac{135{,}687{,}500}{6{,}704{,}125}=20.5'$$

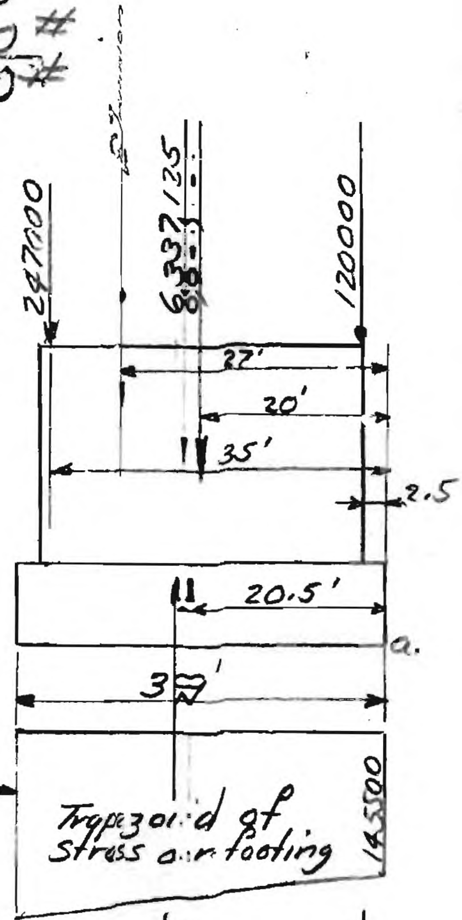
$$K_1=\frac{2P(2l-3l_1)}{l^2}$$

$$=\frac{2(6{,}704{,}125)[2(39)-3(18.5)]}{(39)^2}$$

$$=\frac{13{,}408{,}250[78-55.5]}{1520}=198000\#$$

$$K_2=\frac{2P(2l-3l_2)}{l^2}$$

$$=\frac{13{,}408{,}250(78-61.5)}{1520}=145{,}500$$



Item (2)

Taking moments about "a"

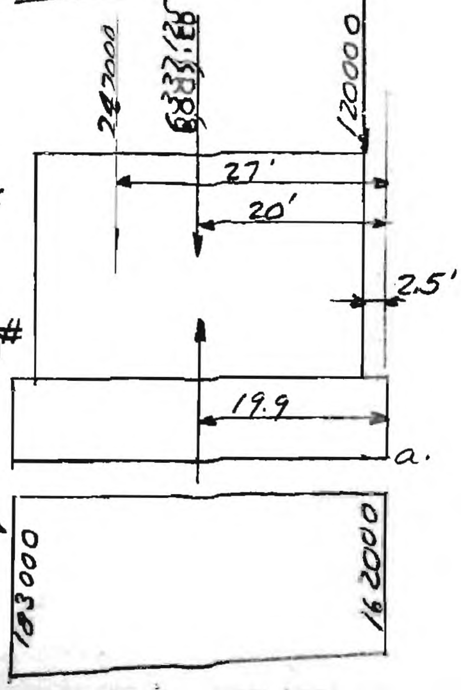
$$\frac{-120000(2.5)-6{,}337{,}125(20)-247000(27)}{6{,}704{,}125}=x$$

$$x=19.9'$$

$$K_1=\frac{13{,}408{,}250[78-3(19.1)]}{1520}=183000\#$$

$$K_2=\frac{13{,}408{,}250[78-3(19.9)]}{1520}=162000\#$$

Note - that values for K_1 & K_2 cover half width of bascule pier



Item (3)

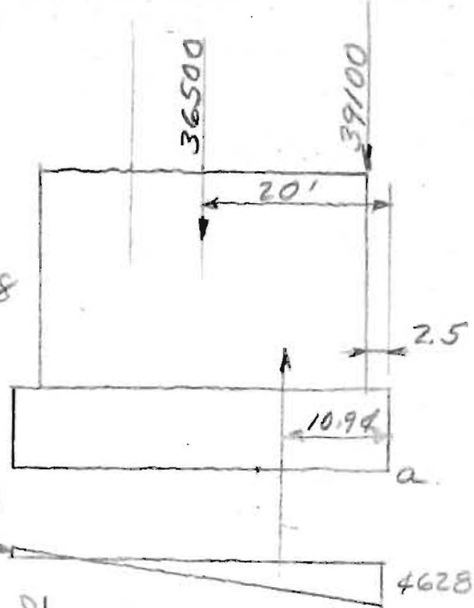
Taking moments about "a"

$$x = \frac{-39100(2.5) - 36500(20)}{75600} = 10.94'$$

$$K_1 = \frac{6(75600)9}{(39)^2} + \frac{75600}{39} = 4628$$

$$= 4628$$

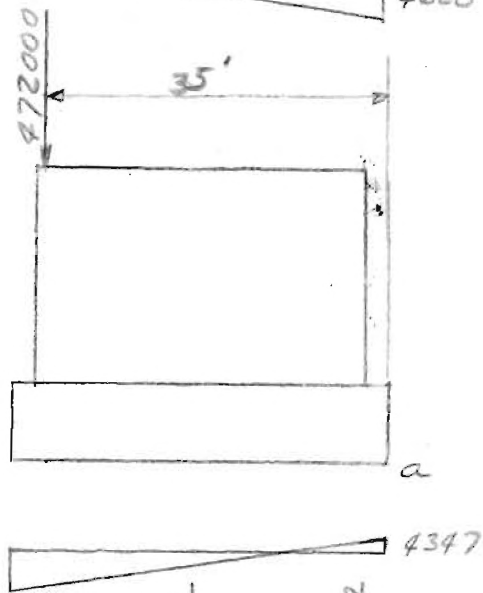
$$K_2 = -\frac{6(75600)9}{(39)^2} + \frac{75600}{39} = -752$$



Item (4)

$$K_1 = \frac{944000(78-12)}{1520} = 41000\#$$

$$K_2 = \frac{944000(78-105)}{1520} = -4347\#$$



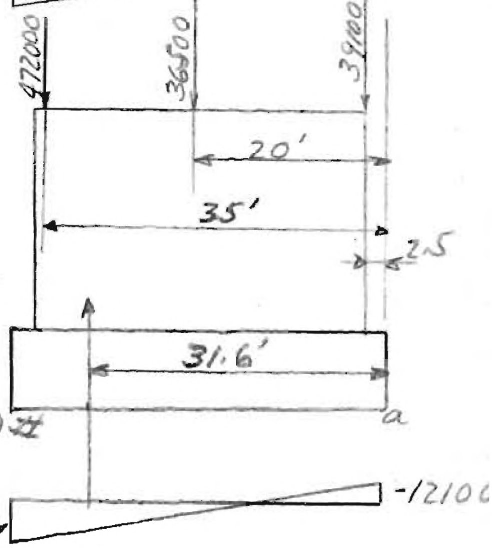
Item (5)

Taking moments about "a"

$$\frac{-39100(2.5) - 36500(20) - 472000(35)}{547600} = 31.6'$$

$$K_1 = \frac{1095200(78-22.2)}{1520} = 40300\#$$

$$K_2 = \frac{1095200(78-94.8)}{1520} = -12100\#$$



Item (6)
Wind $30 \times 60 \times 26 = \frac{46800}{2}$

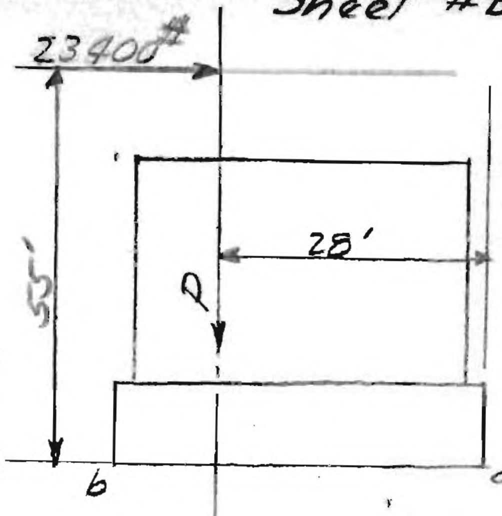
$= 23400$

Taking Moments about a.

$P = \frac{23400(55)}{28} = 46000\#$

$K_1 = \frac{92000(78-33)}{1520} = 2730\#$

$K_2 = \frac{92000(78-84)}{1520} = -363\#$



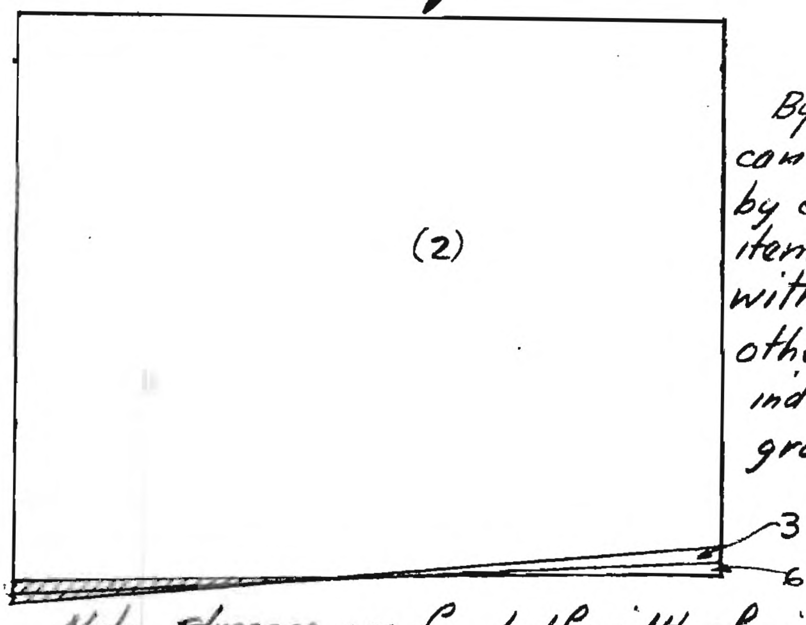
Item (7) Wind from approach
Taking Moments about b

$P = \frac{23400(55)}{11} = 117000\#$

$K_1 = \frac{234000(78-33)}{1520} = 6930\#$

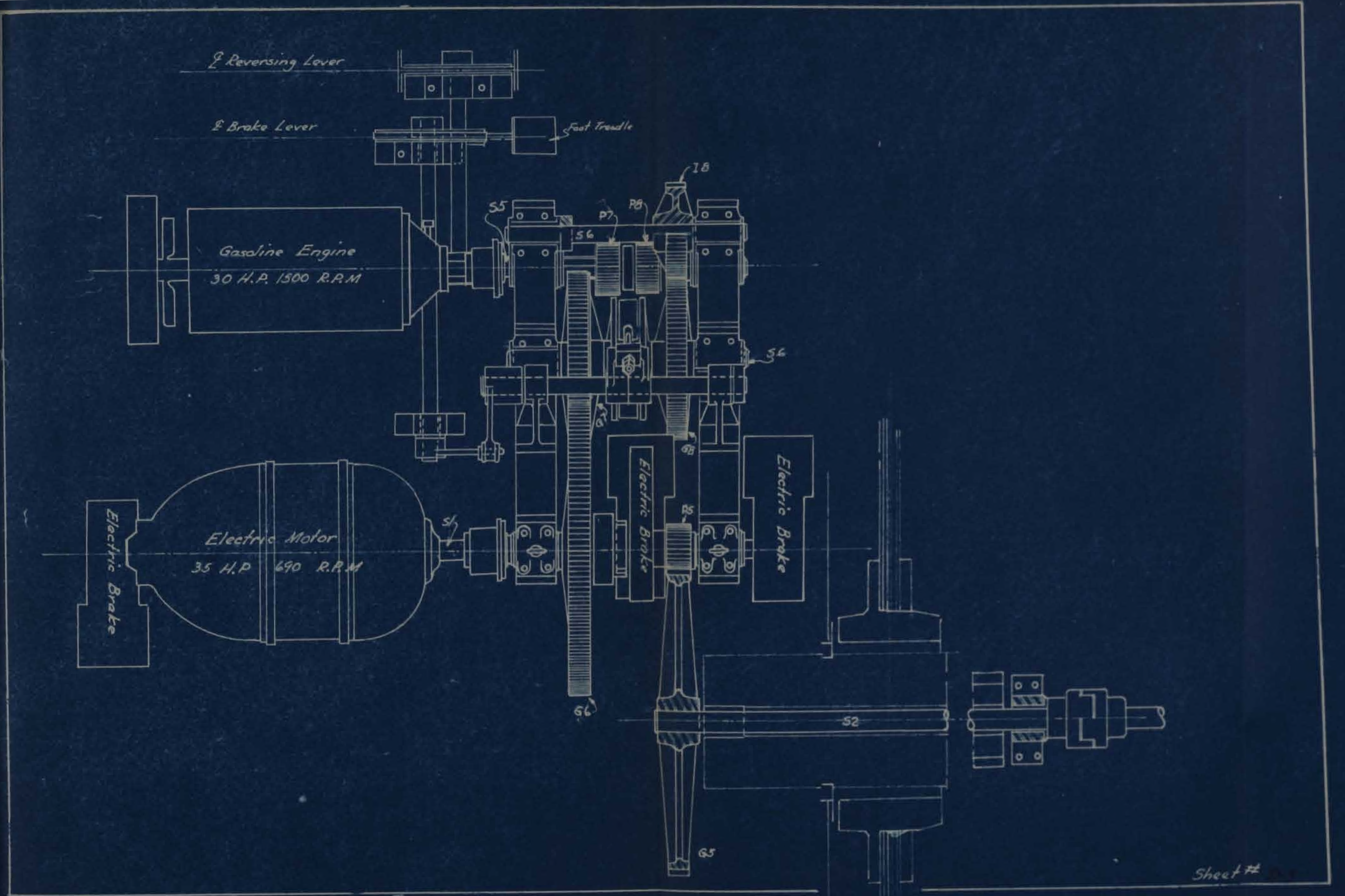
$K_2 = \frac{234000(78-84)}{1520} = -925\#$

In order to obtain the max. stresses the various items will be combined.
[Combination of 2-3 &6]



By observation it can be seen, that by combining either item(1) or item(2) with any of the other items either individually or in groups the pier will be stable

Note- Stresses are for half width of pier for all diagrams



℄ Reversing Lever
℄ Brake Lever
Foot Treadle
Gasoline Engine
30 H.P. 1500 R.P.M
Electric Motor
35 H.P 690 R.P.M
Electric Brake
Electric Brake
Electric Brake
S5
S6
P7
P8
I8
S6
G7
G8
S1
R5
G6
S2
G5
Sheet #

— DESIGN OF OPERATING MACHINERY —

Sheet #M-1

General Data

Distance center to center of trunnions	140'-0"
" " " " " main girders	26'-4"
Rack Radius	20'-0"
Angle of opening	70°-0'
Dead load (moving) one girder only	
River arm	98500#
Rear arm	280000#
Total weight	378500#

Distance center of gravity river arm to trunnion
367200 ÷ 98500 = 37.3'

Distance center of gravity rear arm to trunnion = 13.0'

Area of floor (one girder) 26.5 × 70 = 1855 □'

Distance center of gravity of floor area from trunnion = 35.0'

Coef. of friction (Forged steel trunnions in phosphor bronze bushings) = 15%

Max. starting force at the rack circle – The tangential force at the rack circle applied through the operating pinion must be sufficient to overcome the following resistances.

(1) Inertia of the moving mass
(2) Wind resistance
(3) Frictional resistance

Inertia of the moving mass

m = Equivalent mass at the rack circle
a = Acceleration in feet per second
$F = ma$.

If a weight "W" has its center of gravity c distance from the center of rotation, the equivalent mass reduced to a rack radius "r" = $M = \frac{Wc^2}{32.2 r^2}$, $Mr^2 = \frac{Wc^2}{32.2}$

For River Arm.

$Mr^2 = (98500 \times \overline{37.3}^2) \div 32.2$ = 4,260,000

For Rear Arm

$Mr^2 = (280000 \times \overline{13}^2) \div 32.2$ = 1,470,000

5,730,000

$M = 5730000 \div 400 = 14300$ ft.#

For 70° angular rotation the length of travel on the rack = $[(2\pi)\,20(70°)] \div 360° \doteq 24.5$ ft. approx. Assuming total time of opening at 2 min., the first 30 sec. as the period of acceleration; the last 30 sec. as the time of retardation; & the intermediate 60 sec. as the period of uniform motion.

Uniform velocity $= \dfrac{24.5}{\frac{30}{2}+60+\frac{30}{2}} = .272$ ft. per sec.

Acceleration $= \dfrac{0.272}{15} = .0181$ ft. per sec.^{2}

$F = ma = 2 \times 14300 \times 0.0181 = 518\#$

Wind resistance $15\#$ wind $\dfrac{1855 \times 15 \times 35}{20} = 48800$

Frictional resistance

Load on trunnion Dead load	378500 378500
Wind load on pinion	48800
Total	805800

Frictional force reduced to the periphery of the rack circle. =

$\dfrac{(805800)(9)(0.15)}{240} = 4540\#$

Total Tangential Force at Rack Circle

Inertial	=1036
Wind	48800
Frictional	4540
Total	54376
	Say 54000#

Frict. & Inertional Resist. alone	= 3436
5# wind resistance	= 16260
Total (Say 19700	19696
Frict. & Inertional Resist.	= 3436
10# Wind ristance	= 32520
Total (Say 36000)	= 35956

Design of Rack & Main Drive Pinion (Involute)

Assume cir. pitch, for rack, of 3", a tooth face of 8" & a pinion of 17 teeth

$S = \frac{54000}{3 \times 8 \times .096} = 23400 \#/\square''$

For holding (not operating) against a 15# wind the inertial resistance would be zero. frictional resistance would be

$S = \frac{(48800 - 4500)}{3(8)(.096)} = 20100 \#/\square''$

For operating against a 10# wind

$S = \frac{36000}{3(8)(.096)} = 15600 \#/\square''$

Assuming a 20000 #/□" working stress for forged steel. Stress due to max. wind =

$\frac{2600}{20000} = 13\%$ over stress (which is not excessive)

Number of teeth in a full rack cirle of 20 ft. radius = $[20 \times 3.14 \times 12] \div 3 = 252$

∴ Tooth stress in rack =

$S = \frac{54000}{3 \times 8 \times .15} = 15{,}000 \#/\square''$ which is safe

GEARING DESIGN

Max. tangential force applied at the rack circle by the main drive pinion = 52000# due to wind load

Pitch radius of main drive pinion $= \frac{3 \times 17}{2(3.14)} = 8.1$ inches

Max. torque on the " " " $= 54000 \times 8.1 = 448000$ "#

Due to 10# wind " " $36000 \times 8.1 = 292000$ "#

" " 5# " " " $= 20000 \times 8.1 = 162000$ "#

" " 0# " " " $= 3436 \times 8.1 = 28000$ "#

or 36500 ft.#; 24300 ft.#, 13500 ft.#, 2330 ft.# respectively

Max. lineal velocity of the pinion = .272 ft. per sec

= 16.32 ft. per min.

Circumference of main drive pinion
$= \frac{2 \times 3.14(8.1)}{2} = 4.23$ ft. at the pitch circle

$$\text{Operating Power} = \frac{2\pi(\text{torque})(\text{R.P.M})}{33000} = 26.75$$

There are four sets of pinions & gearing between main drive pinion, and the motorshaft. Since there are two sets of mitre gears, an average efficiency of 90% was assumed
$(.90)^4 = .66$ $100 \div .66 = 150\%$ or 50% extra power to overcome friction loss.

Wind Load (lb)	H.P. Req'd. One Leaf	Torque + 50% ft.# Reduced to Pinion Speed One Leaf
0	1.72	3500
5	9.95	20300
10	17.90	36400
15	40.00	52800
For holding not operating against a 15# wind the values become $\frac{46400}{52000}$ of the last values above	35.80	47000

The motor selected must fulfill the following requirements
(1) It must open the leaf full in the req'd. opening time against a 10# wind
(2) The max. torque developed should be at least 50% greater than req'd. to hold the leaf against a 15# wind
(3) The motor should develope a max. torque 10% greater than is req'd. to operate the leaf against a 10# wind.

A 30 H.P motor @ 580 R.P.M. full load. 440 volt A.C. Induction motor. Full load torque = 188 ft.#
Max. torque = 525 ft.#

$580 \div 3.86 = 150$ = Gear reduction between main pinion and motor shaft.

Motor torques developed

Operating against 10# wind: $\frac{36400}{150} = 343$ ft. #

Holding against 15# wind: $\frac{47000}{150} = 313$ ft. #

TABLE I

TABULATED-REQUIREMENTS OF MOTOR (To Satisfy the above

Conditions as listed above	Full Load Torque	Max Torque	Power
1	343 ft.#		17.90
2		313+50% = 480	
3		343+10% = 377	

DESIGN OF GEAR TRAIN

(See drawings)

Speed of Pinion P1 & Gear G2 = 3.86 R.P.M

" " " P2 " MG3 = $3.86\left(\frac{61}{17}\right)$ = 13.80 " " "

" " " MP3 " MG4 = $13.80\left(\frac{53}{17}\right)$ = 43.20 " " "

" " " MP4 " G5 = $43.20\left(\frac{45}{23}\right)$ = 84.20 " " "

" " " P5 " G6 = $84.2\left(\frac{128}{19}\right)$ = 566.0 " "

Motor was selected at 580 R.P.M which is as close as can be worked out practically

Exact gear reduction between main pinion and the motor shaft $=\left(\frac{61}{17}\right)\left(\frac{53}{17}\right)\left(\frac{45}{23}\right)\left(\frac{128}{19}\right) = 158$

Exact torques required at the motor shaft.

10# wind one leaf: $\frac{36400}{158}$ = 231 ft.#

Holding against 15# wind: $\frac{47000}{158}$ = 298 ft. #

These required torques are less than the computed torques and also less than the tabulated requirements as scheduled in Table I

Gears to be of cast steel, pinions of forged steel;

Gear Design (using 20° Involute teeth)

(G2) $p = 2''$; $f = 5\frac{1}{2}''$; $y = .134$

Pitch Dia. (for 61 tooth gear)

$= \frac{(61)2}{3.14} = 38.8$ inches

$F = 54000 \frac{16.2}{38.8} = 22600\#$

p = circular pitch
f = face width
y = From Lewis Table
F = Tangential pres.

Tooth speed at pitch circle $= 3.86\pi\left(\frac{28.8}{12}\right) = 29.1$ ft. per min.

Efficiency of gears (incl. journal friction) at 93%

$F = \frac{22600}{0.93} = 24300$ incl. friction loss

$s = \frac{24300}{5.5 \times 2 \times .134} = 16500\#/\square''$ which is satisfactory

Tooth stress for 10# wind $= \frac{36000}{52000} \times 16500 = 11400\#/\square''$

(P2) $p = 2''$; $f = 7''$; $y = .096$; No. of teeth = 17

pitch dia. $= 17 \times 2 \div 3.14 = 10.8''$

$F = 24300$

$s = \frac{24300}{7 \times 2 \times .096} = 18000\#/\square''$ O.K

(MG3) No of teeth = 53; $p = 1\frac{3}{4}''$; $f = 3\frac{1}{2}''$; R.P.M = 13.80; $y = .131$

pitch dia. $= \frac{53 \times 1.75}{3.14} = 29.6''$

$F = 24300\left(\frac{10.8}{29.6}\right) = 8850\#$

Tooth speed $= 13.80\pi\left(\frac{33.8}{12}\right) = 12.2$ ft. per sec.

Efficiency of gears (incl. journal friction) at 90%

$F = \frac{8850}{.90} = 9800$ incl. friction loss

$s = \frac{9800}{3.5 \times 1.75 \times .13} = 12300\#/\square''$

(MP3) 17 teeth; $p = 1\frac{3}{4}''$; $f = 3\frac{1}{2}''$; R.P.M = 43.2; $y = .096$

$s = \frac{12300}{1.75 \times 3.5 \times .096} = 21400\#/\square''$

pitch dia. $= \frac{17 \times 1.75}{3.14} = 9.45$

(MS4) Nº of teeth 45, $p = 1\frac{1}{2}$", $f = 3$"; R.P.M = 43.2, $y = .127$

pitch dia. $= \frac{45 \times 1.5}{3.14} = 21.5$"

$F = 9800\left(\frac{9.45}{21.5}\right) = 4300$

Tooth speed $= (43.20)\pi\left(\frac{21.5}{12}\right) = 242.0$ ft. per min.

Eff. of gears = 82%

$F = \frac{4300}{88} = 4880$# incl. friction loss

$s = \frac{4880}{3 \times 1.5 \times .127} = 8500$ #/□"

(MP4) 23 teeth; $p = 1\frac{1}{2}$", $f = 3$, R.P.M = 84.2, $y = .106$

pitch dia $= (23 \times 1.5) \div 3.14 = 11$"

$s = \frac{4880}{1.5 \times 3 \times .106} = 10450$ #/□"

(G5) 128 teeth, $p = 1$", $f = 4$", R.P.M = 84.2; $y = .142$

pitch dia $= (123 \times 1) \div 3.14 = 39.2$"

$F = 4880\left(\frac{11}{39.2}\right) = 1365$#

Tooth speed $= (84.2)\pi\left(\frac{39.2}{12}\right) = 8610$

Eff. = 86 %

$F = 1365 \div 0.88 = 1550$# incl. friction loss

$s = 1550 \div 4 \times 1 \times .142 = 2730$ #/□"

(P5) 19 teeth; $p = 1$"; $f = 4\frac{1}{2}$"; $y = .100$

pitch dia. $= 19 \times 1 \div 3.14 = 6.05$"

$s = \frac{2730}{4.5 \times 1 \times 1} = 6050$ #/□"

(G6) 123 teeth; $p = 1$"; $f = 4$"; $y = .143$

$F = 24300\left(\frac{10.8}{39.2}\right) = 6700$#

pitch dia. $= 123 \times 1 \div 3.14 = 39.2$

$F = 6700 \div .90 = 7440$#

$s = \frac{7440}{4 \times 1 \times .143} = 13000$ #/□"

(G7) 73 teeth; $p = 1$"; $f = 4$"; $y = .138$

$F = 7440$ pitch dia. $= 73 \times 1 \div 3.14 = 23.2$"

$s = 7440 \div 4 \times 1 \times .138 = 13450$

Sheet # M8

(P7) 21 Teeth; $p = 1''$; $f = 4\frac{1}{2}''$; $y = .104$

pitch dia. = $21.1 \div 3.14 = 6.68''$; $F = 7440$ #

$s = 7440 \div 4.5 \times 1 \times .104 = 16500$ #/□"

(P8) 19 Teeth; $p = 1''$; $f = 5\frac{1}{2}''$; $y = .100$

pitch dia. = $19.1 \div 3.14 = 6.08''$

$F = (7440 \times 21) \div 19 = 8210$ #

$s = 9120 \div 5.5 \times 1 \times .1 = 16600$ #/□"

$F = 8210 \div .90 = 9120$ #

(I8) = 51 Teeth; $p = 1$; $f = 5.5$; pitch dia. = $16.234''$; $F = 9120$

(G8) 66 Teeth; $p = 1$, $f = 5.5''$

pitch dia = $21''$

$F = (9120 \times 16.234) \div 21 = 7040$ #

$F = 7040 \div .95 = 7450$ #

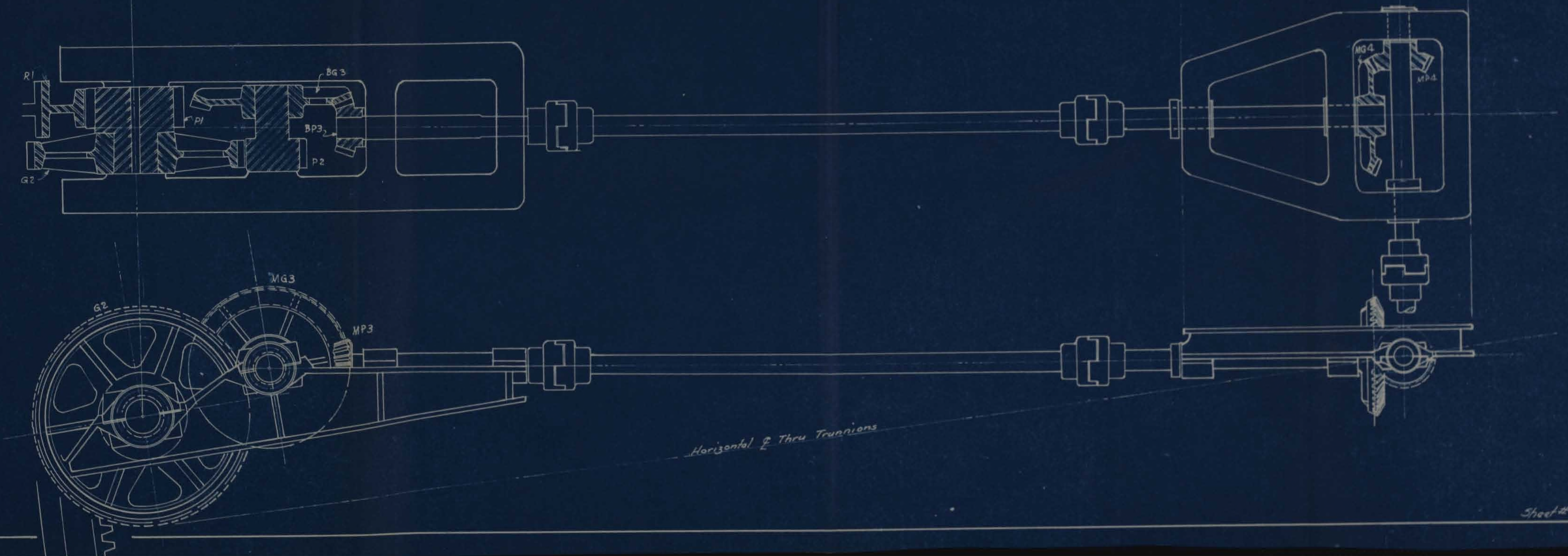
TABLE OF GEAR CALCULATIONS (Type of tooth 20° Involute

Gear Nº	(P) Circular Pitch Inches	Number of Teeth	Pitch Dia. in Inches	R.P.M	Tooth Speed in Ft. per Min.	Est. Eff. Gear & Journal (Per Cent	Total Pressure Without Gear Friction	Total Pressure Including Gear Friction	Torque on Shafting in #	Y; Lewis Formula Factor	Material	Face Width	Unit Stress (lbs.)
R-1	3"	252										8"	
P-1	3"	17	16.2	3.86		.93					Forged Steel	8"	15600
G-2	2"	61	38.8	3.86	29.1	.93	22600	24300	472000	.134	Cast "	5½"	16500
P-2	2"	17	10.8	13.80	39.0		22600	24300	131500	.096	Forged "	7"	18000
MG-3	1¾"	53	29.6	13.80	12.2	.90	8850	9800	145000	.131	Cast "	3½"	12300
MP-3	1¾"	17	9.45	43.20	12.2		8850	9800	46300		Forged "	3½"	21400
MG4	1½	45	21.5	43.20	2420	.85	4300	4880	52400	.127	Cast "	3"	8500
MP4	1½	23	11.0	84.20	242.0		4300	4880	26900	.106	Forged "	3"	10450
G5	1	128	39.2	84.20	8610	.90	1365	1550	30400	.142	Cast "	4"	2730
P5	1	19	6.05	566.0	8610	.90	1365	1550	4690	.100	Forged "	4½"	6050
G6	1	123	39.2	566.0	-	.90	6700	7440	14500	.143	Cast "	4"	13000
G7	1	73	23.2		-	.90	6700	7440	86200	.138	Forged "	4"	13450
P7	1	21	6.68	1400	-	.90	6700	7440	24900	.104	Cast "	4½"	16500
P8	1	19	6.08	1400	-	.90	8210	9120	27400	.100	Forged "	5½"	16500
G8	1	66	21.00	-	-	.95	7040	7450					
I8	1	51	16.23	-	-		8210	9120					

Check on motor shaft P5, $\frac{4690}{12} = 390$ ft. #

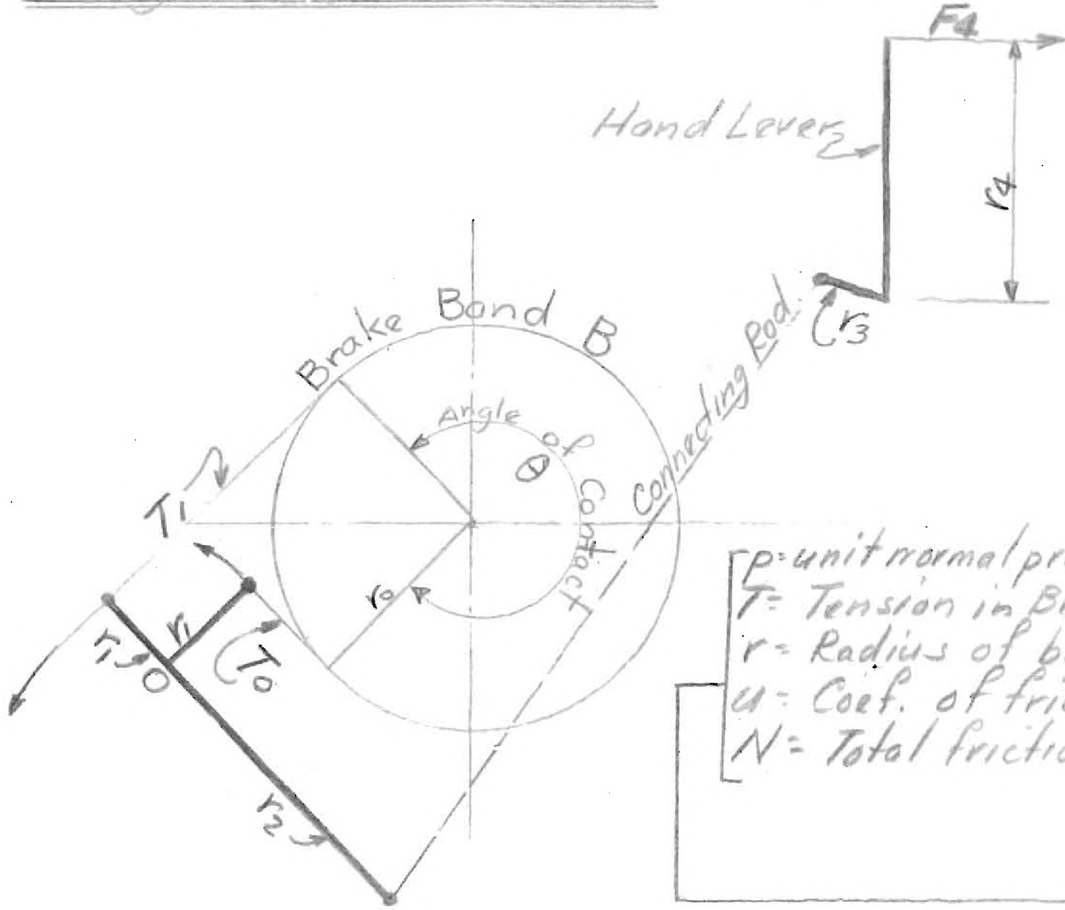
390 × gear ratio = 390 × 158 = 61600 ft. # approx.

Sheet # M9


R1
BG3
P1
BP3
P2
G2
MG4
MP4
MG3
G2
MP3
Horizontal ℄ Thru Trunnions

Sheet #

Design of Handbrakes



p = unit normal pres. at any pt.
T = Tension in Brake Band
r = Radius of brake cyl.
u = Coef. of friction
N = Total friction $\Sigma upds$

Applying Force "F" to the lever as shown above

$(T_0 + T_1)r_1 = \dfrac{F r_4 r_2}{r_3}$, $T_0 + T_1 = \dfrac{F r_4 r_2}{r_3 r_1}$

Assuming belt is about to slip;-

diagram A

Diagram B is a free body diagram, taken from diagram A, of an element of brake belt of length ds. The forces acting on this element are T & T+ dT at the ends & reaction of the brake cylinder.

$p = \dfrac{T}{r}$

Refer to diagram B

$dN = upds = dT$

$ds = r d\theta$

$\dfrac{dT}{T} = \dfrac{upds}{T} = \dfrac{uprd\theta}{T} = \dfrac{ud\theta(pr)}{T} = ud\theta$

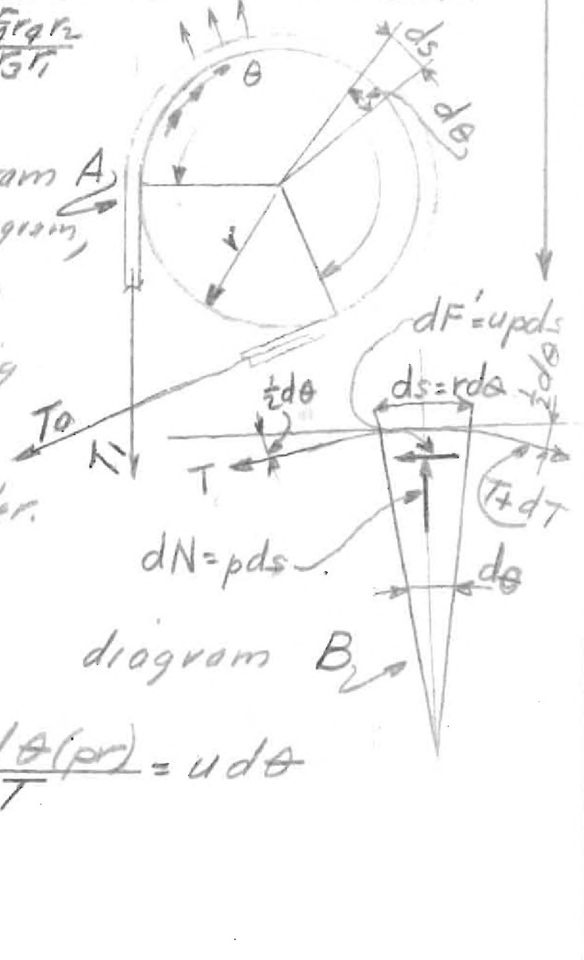


diagram B

<u>Design of Hand Brake Cont</u>

$\frac{dT}{T} = u\,d\theta$ Integrating this equation the relation between T_1 & T_0 may be found

$$\int_{T_1}^{T_0} \frac{dT}{T} = \int_0^{\theta} u\,d\theta$$

$$\log_e \frac{T_0}{T_1} = u\theta \quad \text{or} \quad \frac{T_0}{T_1} = e^{u\theta}$$

$$T_0 = T_1 e^{u\theta}$$

where e is the base of naperian logarithms 2.71828

u = Coefficient of friction

θ = The angle of contact in radians

Substituting in our original equation

$$T_0 + T_1 = \frac{F_4 r_4 r_2}{r_3 r_1} = T_0(1 + e^{\theta u}) = \frac{F_4 r_4 r_2}{r_3 r_1}$$

Torque on brake band $= (T_1 - T_0) r_0 = T_0 (e^{u\theta} - 1) r_0$

by subst.

$$\text{Braking Torque} = \left(\frac{F r_4 r_2}{r_1 r_3}\right)\left(\frac{e^{u\theta} - 1}{e^{u\theta} + 1}\right) r_0$$

Values of $e^{u\theta}$ for various angles of contact are given below (based on assumed coef. of friction of 20 percent)

Values of θ in degrees	Value of $e^{u\theta} = e^{0.20\theta}$
140	1.63
160	1.74
180	1.87
200	2.01
220	2.16
240	2.31
270	2.57
300	2.85

Braking torque necessary to hold the moving leaf against a 15# wind =

(48800 − 4500) 19.57 = 876,951 in# (approx) on the main pinion shaft

Reduced to the speed of shaft 56

$876951 \times \frac{73}{123} - 52,000$ in#

Design of Hand Brakes Ctd.

Assuming a maximum pull at the end of the brake lever equal to 100#

Taking

$r_1 = 3''$

$r_2 = 24''$

$r_3 = 3''$

$r_4 = 60''$

$r_0 = 9''$

$e^{\mu\theta} = 2.57$

$F = 100\#$

$$\text{Braking Torque} = \left[\frac{(100)60(24)}{3(3)}\right]\left[\frac{2.57-1}{2.57+1}\right]9 = 63,400 \text{ in}\#$$

which is satisfactory.

Centre Lock Mechanism

Total shearing stress (2 Girders) transmitted by the centre lock = 20000#

Assuming coef. of friction of 30%

pull on centre lock = .30(20000) = 6000#

Assuming length of time req'd. in locking or unlocking the bridge as 10 seconds

Power developed = (Travel of locking pins = $6\frac{1}{2}''$)

$$\frac{6000 \times 6.5}{10} = 3900 \text{ in.\# per sec.}$$

$$\text{or } \frac{(3900)(60)}{12} = 19500 \text{ ft.\# per min.}$$

$$\frac{19500}{33000} = 0.59 \text{ H.P.}$$

Assuming overall eff. of 60%

$$\frac{.59}{.60} = .985 \text{ H.P. or approx. } 1.0 \text{ H.P.}$$

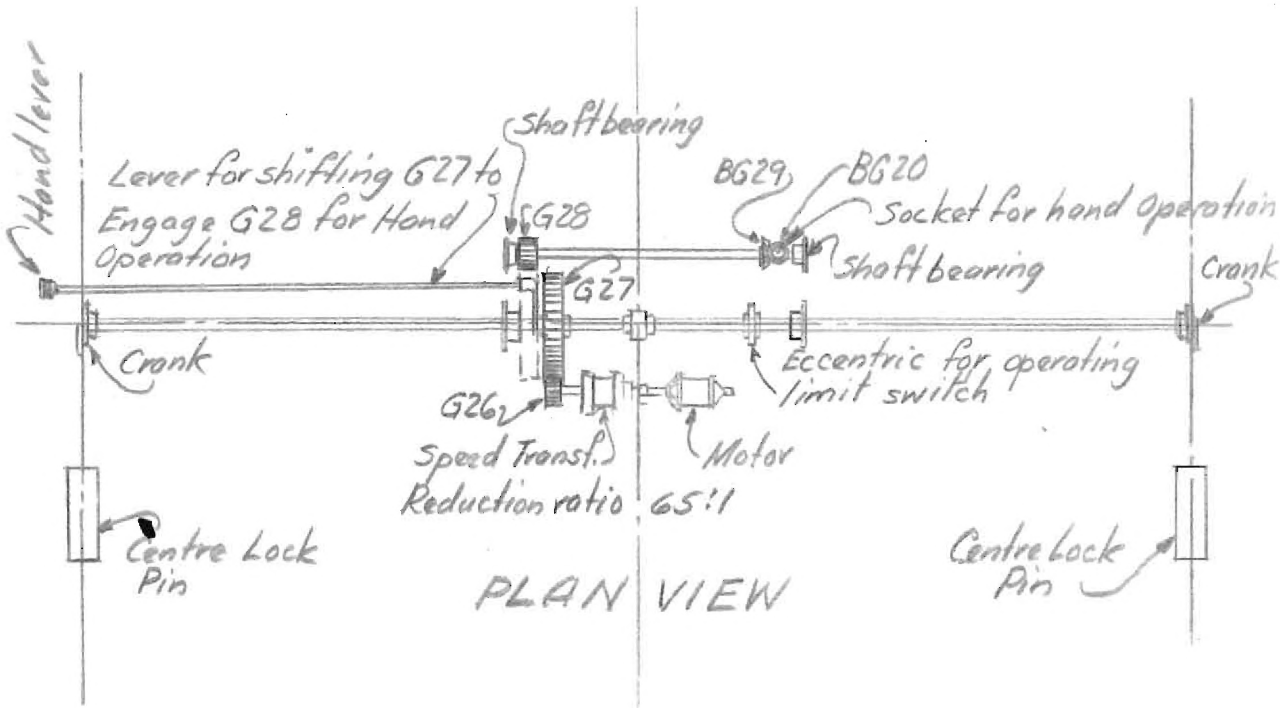
An electric motor will be used placed at the end of the moving leaf.

Inasmuch as there is very little difference in the cost between a 1 H.P & a 3 H.P. electric motor and, too, since an excess of power is desirable in this particular case, an electric motor of 3 H.P. will be selected. A.C. running at a full load speed of 1,125 R.P.M.

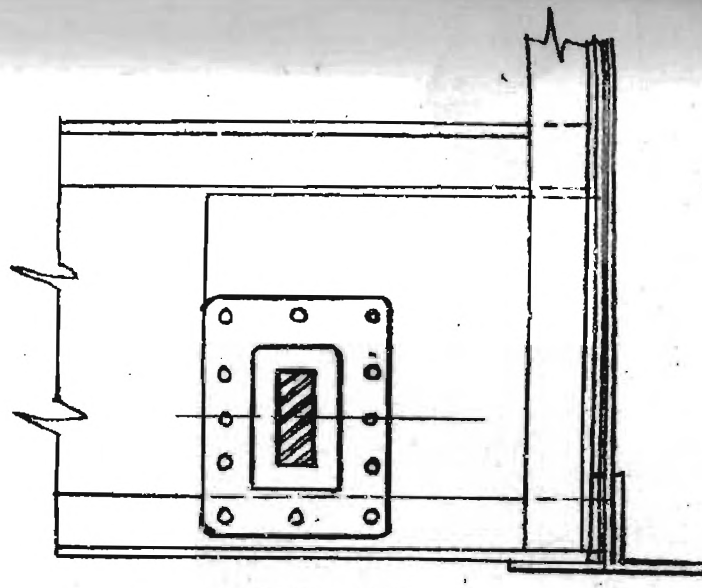
The complete operation of locking or unlocking the bridge requires a one-half revolution of the crank shaft and requires a time interval of 10 sec.. During this time the motor makes approx. $1125 \times \frac{10}{60} = 188$ rev.

Gear reduction $\therefore = 188 \div \frac{1}{2} = 376$

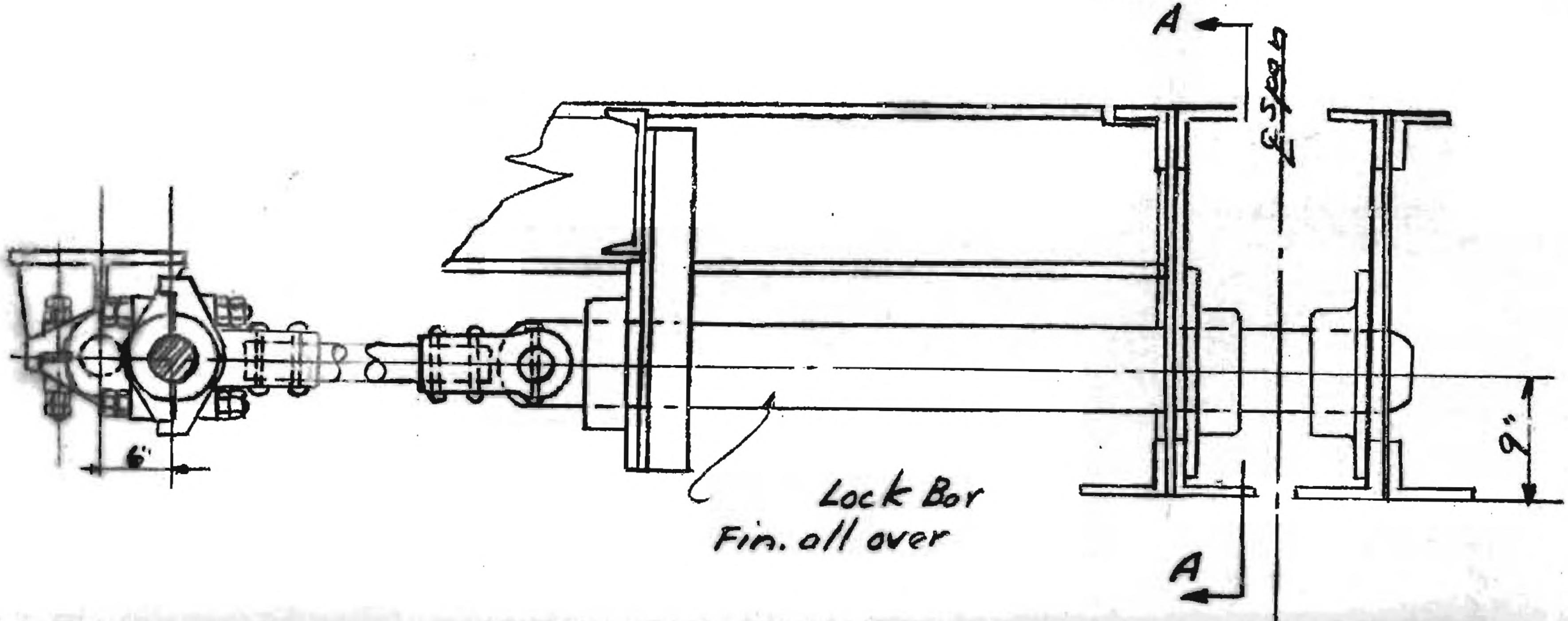
A spur gear of a reduction of 3 to 1 together with a speed transformer of a ratio of 65:1 will be adopted.



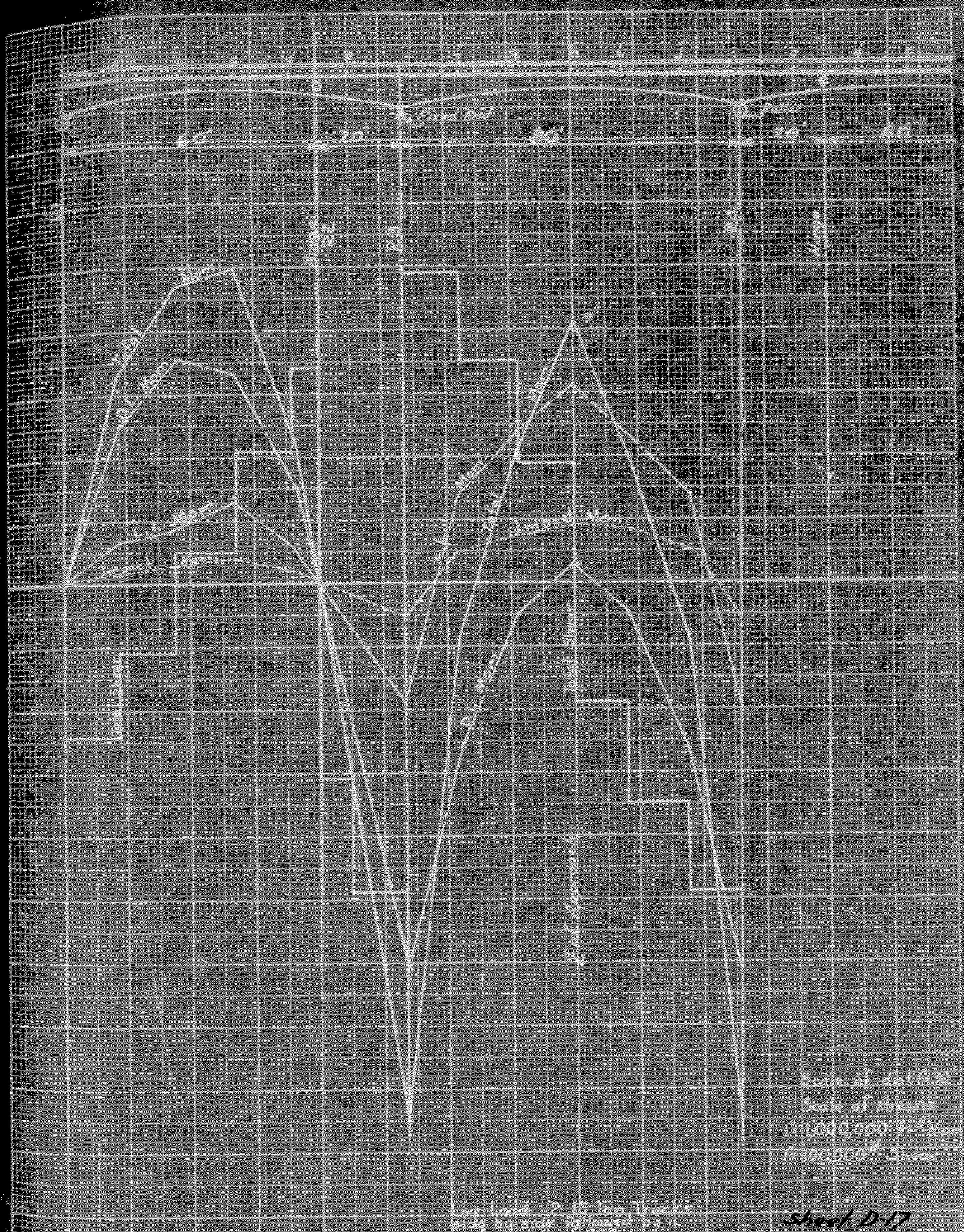
PLAN VIEW



SECT. AA



DETAIL OF CENTER LOCK

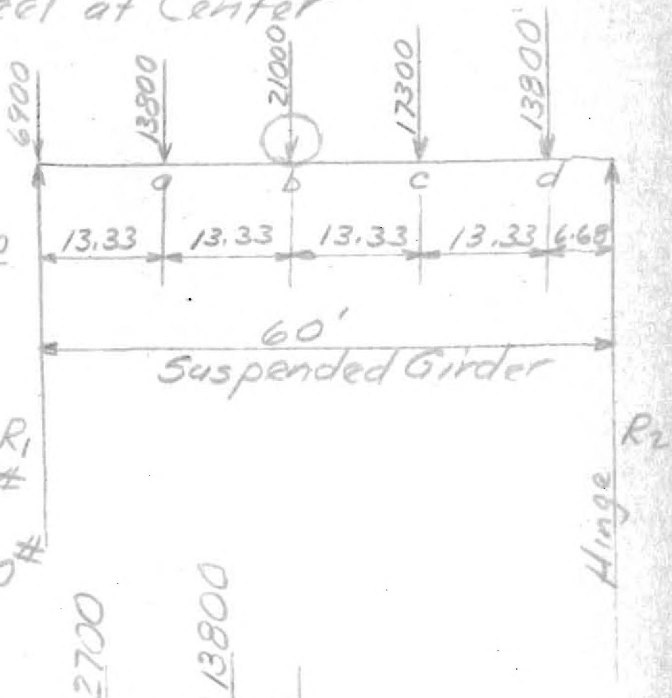

Fixed End
60'
20'
80'
20'
60'
Total
D.L. Mom.
L.L. Mom.
Impact Mom.
Scale of stresses
1"=1,000,000 ft.# Mom.
1"=100,000# Shear
Live Load 2-15 Ton Trucks
side by side followed by
uniform L.L. at 80%

Sheet D-17

Design Of Approach Spans

Live Load.

End Span - Large Wheel at Center
(Suspended Girder)



R1

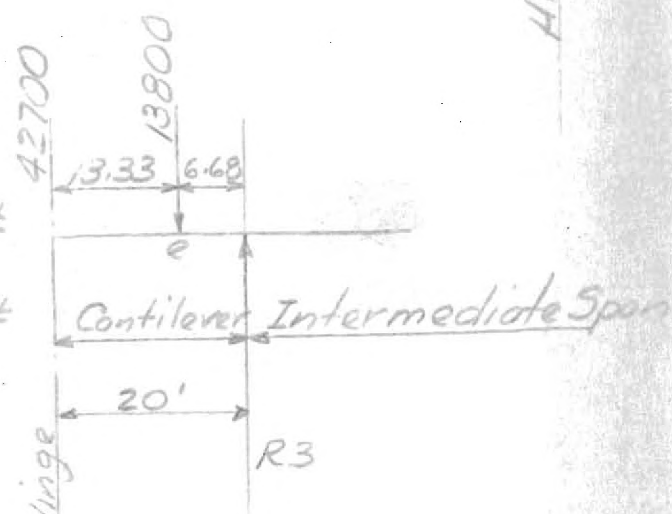
$6900(60) + 13800(46.67) + 21000(33.34) + 17300(20) + 13800(6.68)$

$= \dfrac{414000 + 644000 + 700000 + 346000 + 92300}{72800}$

$R1 = \dfrac{2,196,300}{72800} = 30100\#$

Hinge $= 72800 - 30100 = 42700\#$

R3 $= 42700 + 13800 = 56500\#$



Cant. Mom. L.L

$42700(20) + 13800(6.68)$

$= 854000 + 92100 = 946100$ ft.#

$Ma = 30100(13.33) - (6900)(13.33)$

$= 401000 - 92000 = 309000$ ft.#

$Mb = 30100(26.66) - 6900(26.66) - 13800(13.33)$

$= 802000 - 184000 - 184000$

$= 434000$ ft.#

$Md = -42700(6.68) = 285000$ ft.#

$Mc = -42700(20) - 13800(13.33) = 854000 - 184000 = 670000$ ft

L.L. Shears

Between R_1 & a	30100 - 6900	=	23200
a " b	23200 - 13800	=	9400
Hinge to d		=	42700
d to c	42700 - 13800	=	28900
c " b	28900 - 17300	=	11600

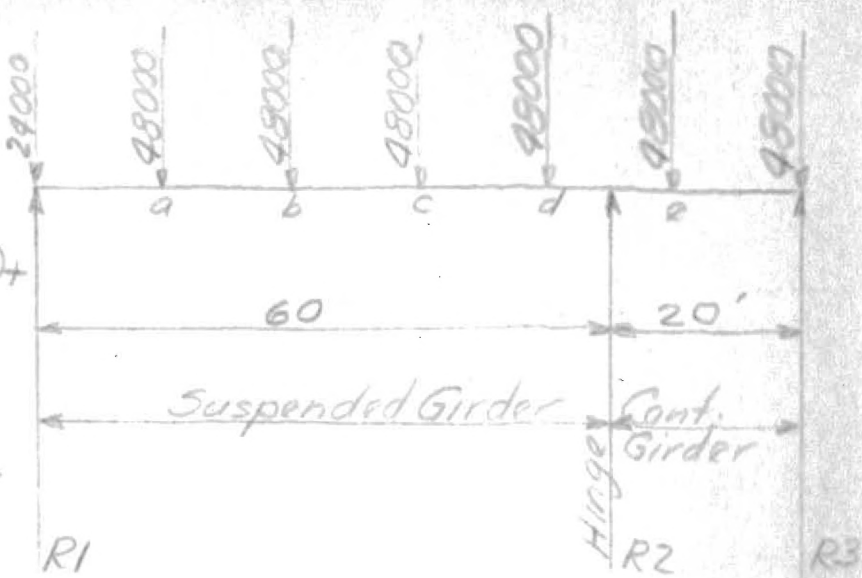
End Span D.L.
(Suspended Girder)

H.R. & Girder

$= 1400 \times 13.33 = 18{,}700$

$170 \times 13 \times 13.33 = \underline{29500}$

$48200\#$



$$\frac{24000(60)+48000(46.67)+48000(33.34)+}{60}$$

$$\frac{48000(20.01)+48000(6.68)}{60}$$

$$= \frac{1440000+2{,}240{,}000+1{,}600{,}000}{60} +$$

$$\frac{960000+320000}{60}$$

$$R1 = \frac{6{,}560{,}000}{60} = 109400\#$$

$$R2 = 216000 - 109400 = 106{,}600\#$$

$M_a = 109{,}400(13.33) = 1{,}460{,}000$ ft.#

$M_b = 109{,}400(26.66) - 48000(13.33) = 2{,}280{,}000$ "

$M_c = 109{,}400(39.99) - 48000(26.66) - 48000(13.33) = 2{,}448{,}000$ "

$M_d = 106{,}600(6.68) = 713{,}000$ "

Cantilever Moms.

$M_{R3} = 106{,}600(20) + 48000(13.33 = 2{,}772{,}000$ ft.#

D.L. Shears

R1 to a	109400
a " b	61400
b " c	13400
R_2 " d	106600

Sheet A-3

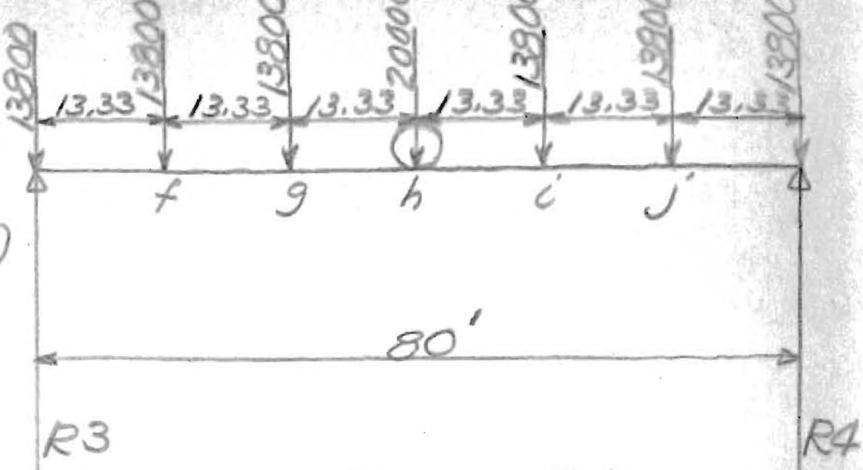
Summary of Stresses
Suspended Girder

R-1	a		b		c		d	
Shears D.L.	109400		61400		13400		106600	
" L.L	23300		9400		28900		42700	
I	6990		2820		8670		12810	
Total	139690		73620		50970		162110	
Sect. Reqd. 10000#/Gr.	13.96□"		7.36□		5.09□"		16.21□"	
Web used	$69 \times \frac{3}{8} = 25.9$		$52 \times \frac{3}{8} = 19.5$		$46 \times \frac{3}{8} = 17.25$		$53 \times \frac{3}{8} = 19.8$	
Moms. ft. #								
" D.L.	1251000		1786000		1617000		788000	
" L.L.	309000		434000		670000		285000	
" I	92700		130200		201000		85500	
" Total	1652700		2350000		2488000		1158000	
Eff. Depth	5.46'		4.03'		3.53'		4.12'	
Top Flange Stress	302700		583000		705000		281000	
Bott " "	305000		583000		705000		281000	
Flange Reqd.	net. 19.06□		36.4□ net		44.1□" net		17.56□ net	
Sect. Used	Gr.	Net	Gr.	Net	Gr.	Net	Gr.	Net
1/8 Web	3.23	3.23	2.43	2.43	2.15	2.15	2.48	2.48
$2\angle s\ 6 \times 6 \times \frac{3}{4}$"	16.88	13.88	16.88	13.88	16.88	13.88	16.88	13.88
1 PL 13×1"	13.00	11.00	13.00	11.00	13.00	11.00	13.00	11.00
1 PL 13×1"			13.00	11.00	13.00	11.00		
2 PLs 8"×5/8" (Side PLs)					10.00	7.00		
	33.11	28.11	45.31	38.31	55.03	45.03	32.36	27.36
Flange Sections								

Center Span

L.L.

$R3 = 13800(80) + 13800(66.67) + 13800(53.34) + 20000(40) + 13800(26.68) + 13800(13.33) \div 80$



$$\frac{1{,}104{,}000 + 920000 + 735000 + 800000 + 369000 + 184000}{80} = R3$$

$$R3 = \frac{4{,}112{,}000}{80} = 51400\#$$

$R4 = 51400$

$M_f = 51400(13.33) = 685000$ ft#

$M_g = 51400(26.66) - 13800(13.33) = 1189800$ " "

$M_h = 51400(40) - 13800(26.66) - 13800(13.33) = 1502500$ " "

L.L. Shears

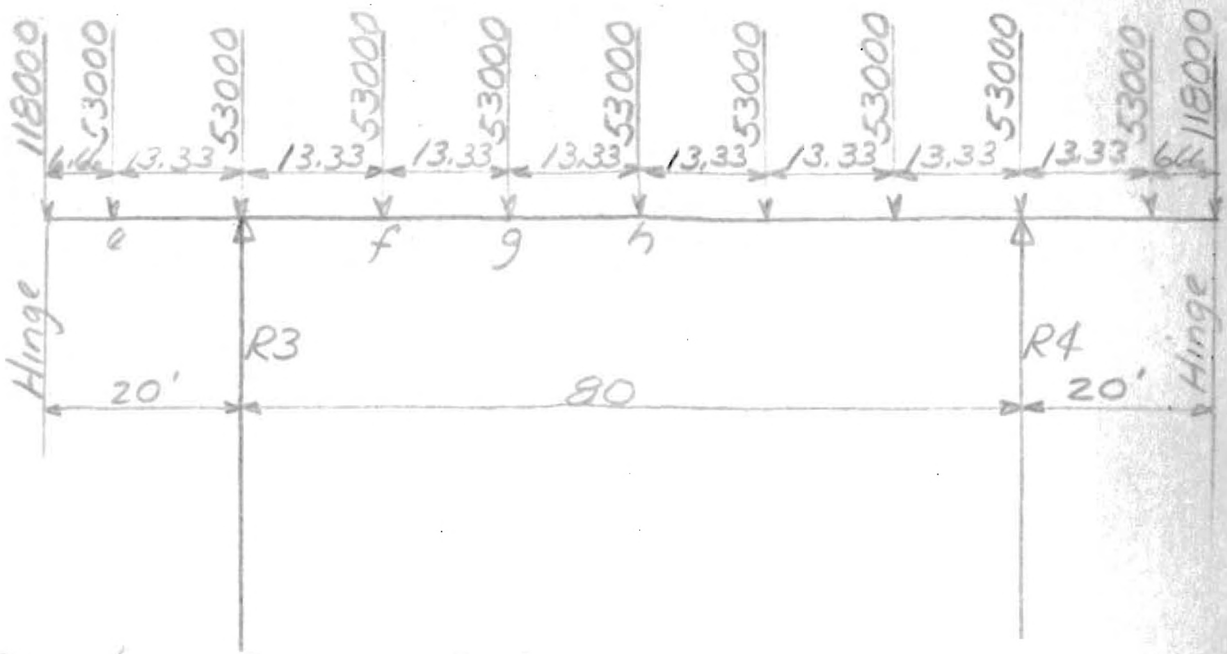
R3 to f = 51400

f to g = 51400 − 13800 = 37600

g " h = 37600 − 13800 = 23800

D.L. Shears (See Diagram Sheet A-5)

e to R3	=	171000
R3 " f	=	159000
f " g	=	106000
g " h	=	53000



Center Span D.L.

R3 = R4

= 118000 + 4.5(53000) = 356000 #

MR3 = 118000(20) - 53000(13.33) = 3026500 ft.#

Mf = 118000(33.33) - 53000(26.66) - 53000(13.33) + 356500(13.33) = -1291000 ft.#

Mg = -118000(46.66) - 53000(40) - 53000(26.66) - 53000(13.33) + 356000(26.66) = - 231000 ft.#

Mh = -118000(60) - 53000(53.32) - 5300 0(40) - 53000(26.66) - 53000(13.33) + 356000(40) = 109000 ft.#

See Sheet A6 for Tabulation of Stresses

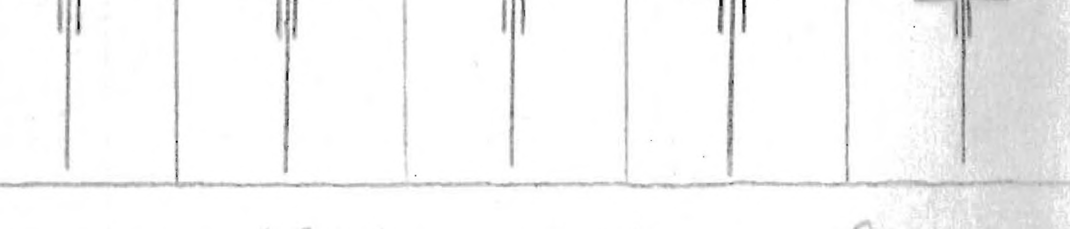
See Sheets A1 & A2 for cantilever Stresses

Summary of Stresses
Intermediate Girder

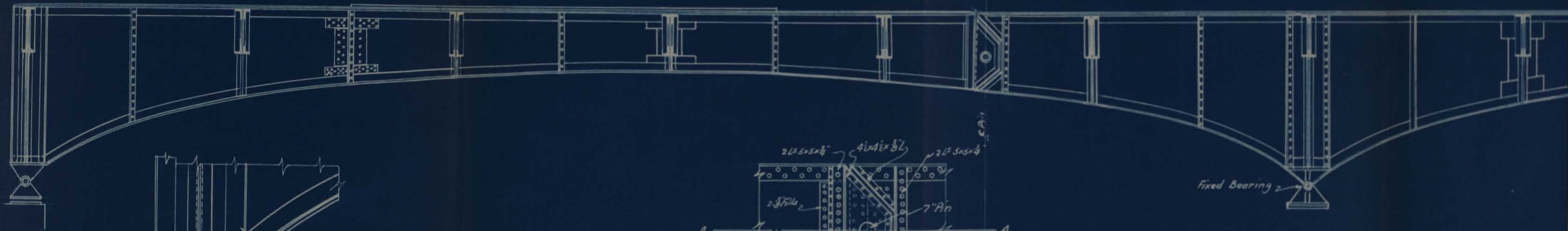
	e	R3	f	g	h
Shear D.L.	118000	171000	159000	106000	53000
" L.L		56500	51400	37600	23800
" I	35400	16950	15420	31800	15900
" Total	153400	244450	225820	175400	92700
Sect. Reqd.	15.34 □″ Gr.	24.44 □″ Gr.	22.58 □″ Gr.	17.54 □″ Gr.	9.27 □″ Gr.
Web Used	$66 \times \frac{3}{8} = 24.75$	$114 \times \frac{3}{8} = 42.75$	$66 \times \frac{3}{8} = 24.75$	$48 \times \frac{3}{8} = 18$ □″	$46 \times \frac{3}{8} = 17.25$
Moms. ft.#					
" D.L.		−3026500	−1291000	−231000	+109000
" L.L		−946000	+685000	±1189800	±1502500
" I		−283800	+205500	+356940	+450750
" Total		−4256300	−400500	±1015740	±2062250
Eff. Depth	5.21	9.21	5.21	7.08	3.54
Top Flg. Stress		462000	76800	143500	583000
Bott " "		464000	77000	143000	583000
Flange Reqd.		29 □″ net	4.81 □″ net	8.94 □″ net	36.43 □″ net

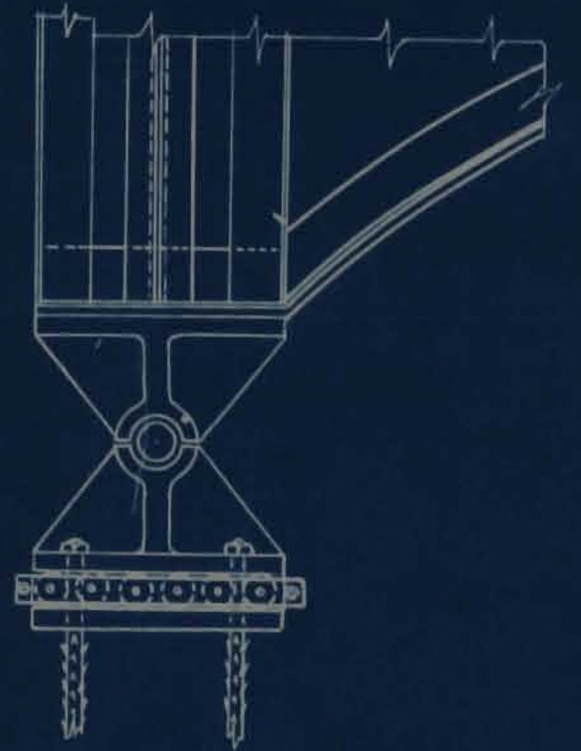
Sect Used	e Gr.	e Net	R3 Gr.	R3 Net	f Gr.	f Net	g Gr.	g Net	h Gr.	h Net
1/8 Web	3.09	3.09	5.35	5.35	3.09	3.09	2.25	2.25	2.16	2.16
2 Ls 6×6×3/4″	16.88	13.88	16.88	13.88	16.88	13.88	16.88	13.88	16.88	13.88
1 Pl 13×1″	13.00	11.00	13.00	11.00	13.00	11.00	13.00	11.00	13.00	11.00
1 Pl 13×1″									13.00	11.00
	32.97	27.97	35.23	30.23	33.97	27.97	32.13	27.13	45.04	38.04

Flange Sect.

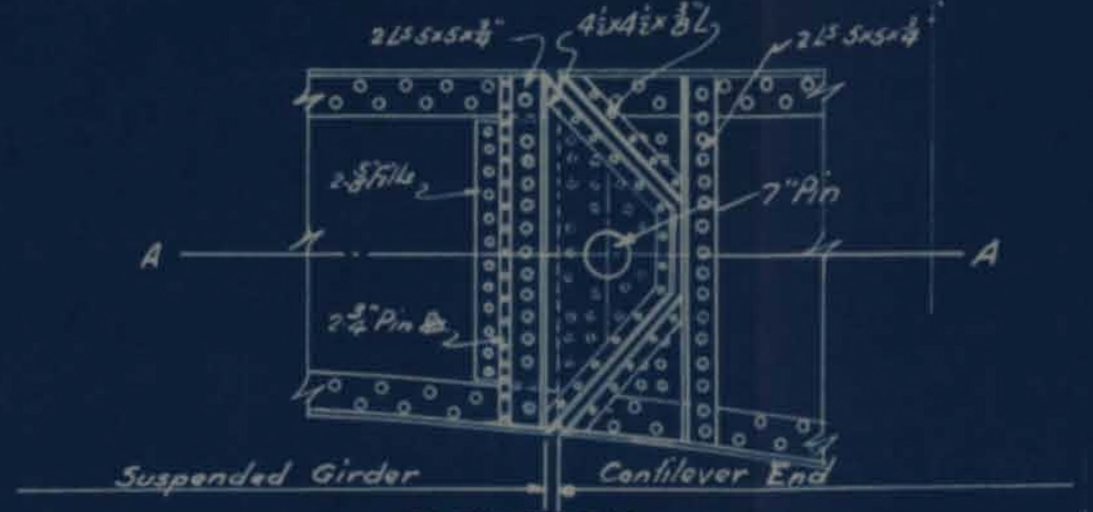


From Specifications thickness of web to be not less than 1/160 of unsupported dist. between flange Ls $= \frac{3.54 \cdot 12}{160} = .265''$
$\therefore$ 3/8″ Pl as used satisfies the specifications





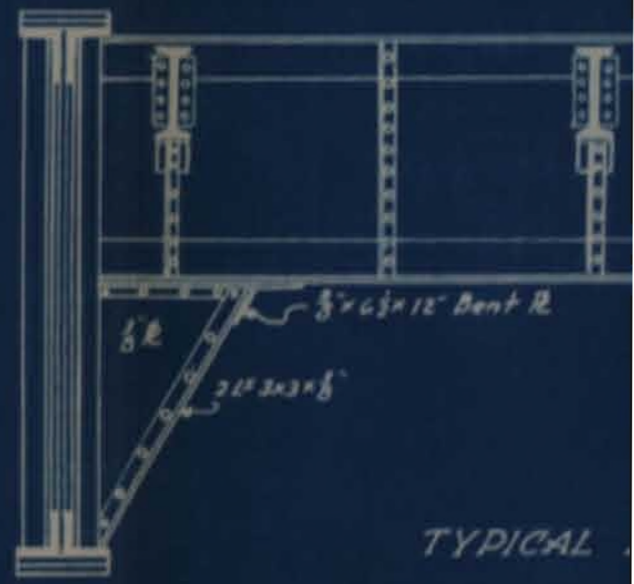
ROLLER BEARING AT
ABUTMENT & BASCULE PIER



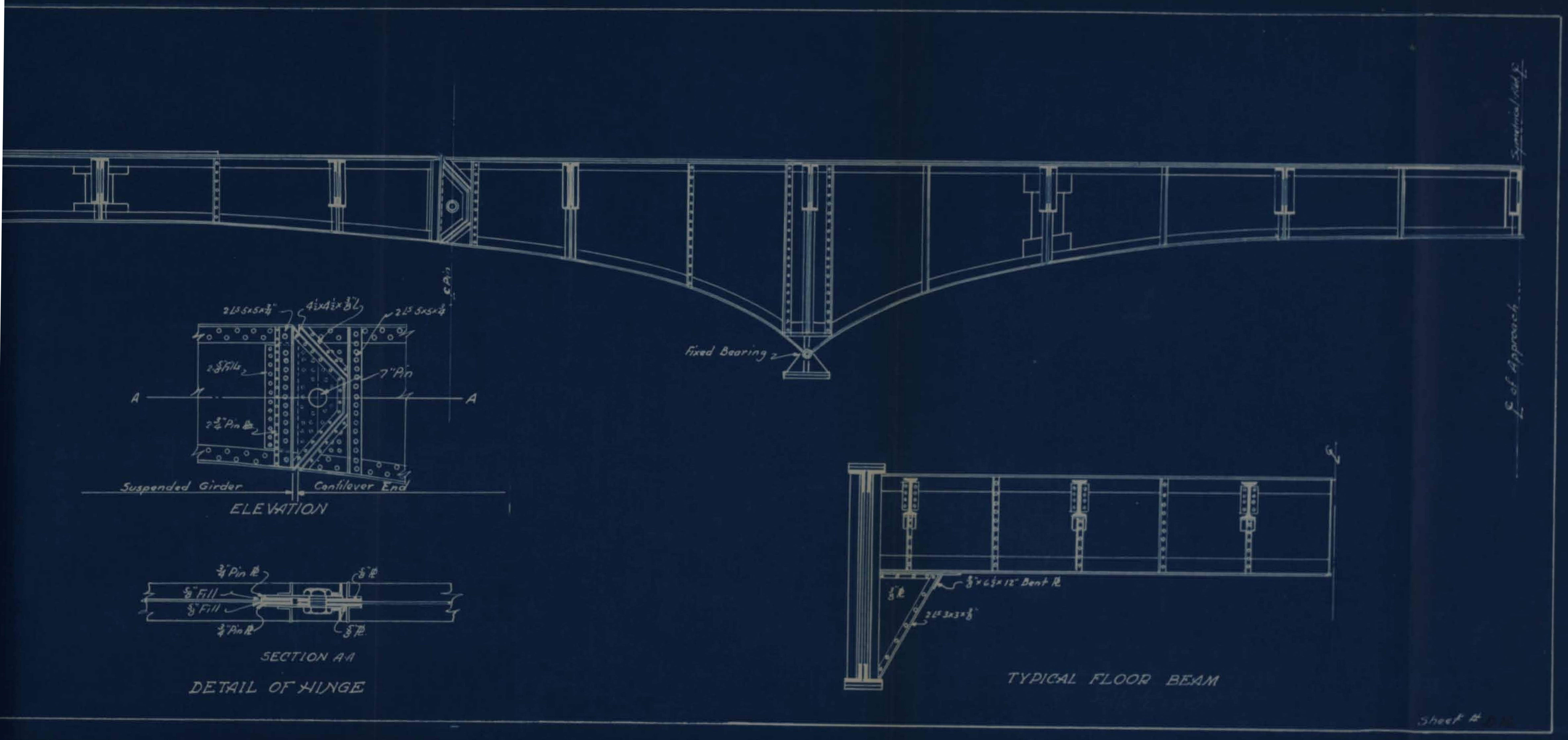
ELEVATION

SECTION A-A

DETAIL OF HINGE



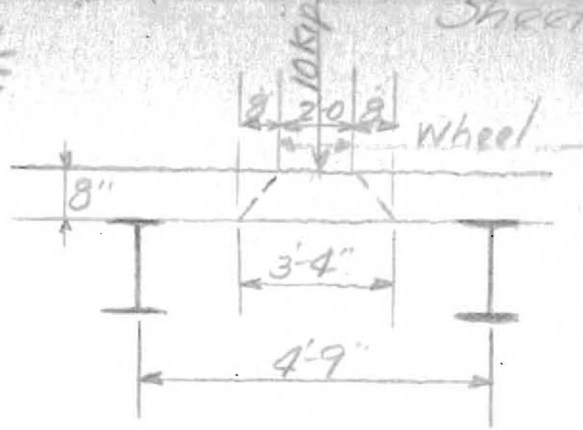
TYPICAL



Fixed Bearing
7" Pin
A
A
Suspended Girder
Cantilever End
ELEVATION
SECTION A-A
DETAIL OF HINGE
TYPICAL FLOOR BEAM
℄ of Approach
Sheet #

Concrete Deck

Span 4'-9"
Dead load $100^{\#}/_{\square}'$
Paving $\underline{40^{\#}/_{\square}'}$
140



$3.33 \times 3.33 = 11.11^{\square}$

$\frac{10000}{11.11} = 900^{\#}/_{\square}'$

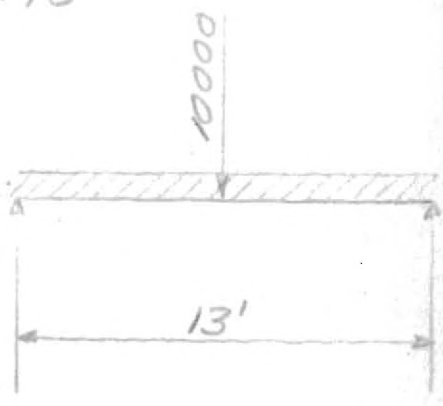
$900 + 140 = 1040$

$M = 1040(4.5)(4.5)1.5 = 31590\ ''\# \quad +30\%\ I = 41067''\#$

$d = \sqrt{\frac{41067}{12 \cdot 113}} = 5'' + \quad T = 7''$

A_s at Center $= \frac{41067}{16000 \cdot \frac{7}{8} \cdot 6} = .48^{\square}''$

Stringers: Span 13'



$M_c = 5000 \cdot 6.5 = \quad 32500$ ft.#
$M_u = \frac{715 \cdot 13 \cdot 13}{8} = \quad \underline{15100\ '' \ ''}$
$\quad 47600\ ''\ ''$
$47600 \times 12 = \quad \underline{571200}$ in#
Shear $= 6.5 \times 715 = 4650^{\#}$
$\quad \underline{5000^{\#}}$
$\quad 9650^{\#}$

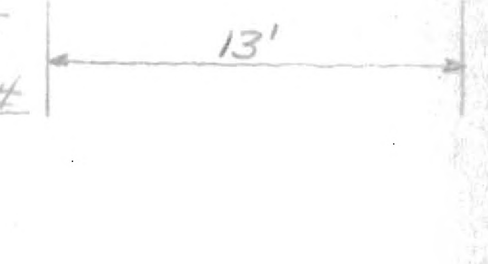
Sect. Mod. $\frac{571200}{16000} = 35.6$

Use 12" C.I. @ 31.8#

FLOOR BEAMS

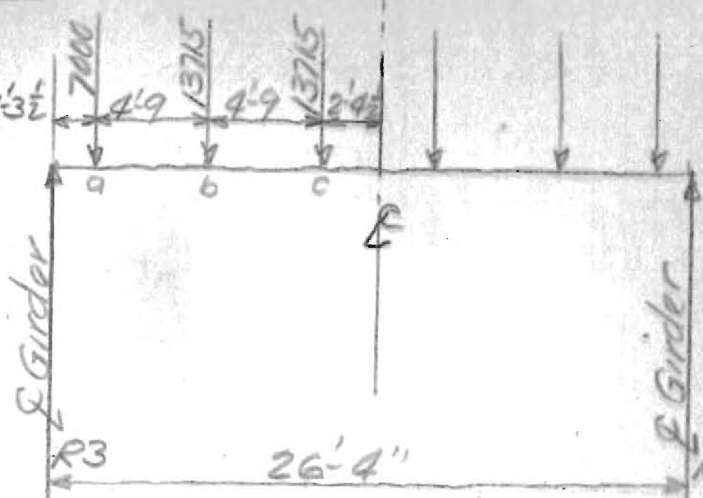
L.L. Rear wheels 6'-0 apart, 10000# on ea. wheel
Uniform load = 80×4.75 = 380# per ft. of stringer
Reaction = 380×6.5 = 2470#

$R1 = 10000 + \frac{5000 \times 3.33}{13.33}$
$= 10000 + 1245 = 11245$
$R1 = 11245 + 2470 = 13715^{\#}$



FLOOR BMS. CTD.

$R3 = 13715 + 13715 + 7000$
$= 34430\#$



Moments

$M_c = 34430(10.75) - 7000(9.5) - 13715(4.75) = 248977 \text{ ft}\#$

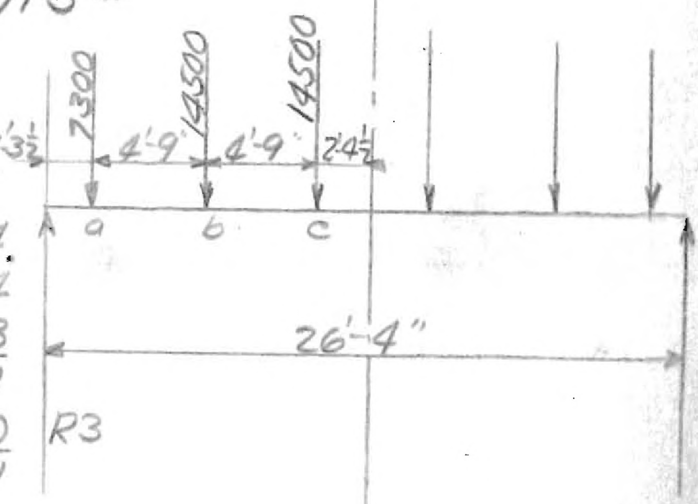
$M_b = 34430(6) - 7000(4.75) = 173330 \text{ ft}\#$

$M_a = 34430(1.25) = 43038 \text{ ft}\#$

L.L. Shears

R_3 to a = $34430\#$
a to b = $27430\#$
b to c = $13715\#$

DEAD LOADS



Paving $140\#/\square'$
$140 \times 4.75 \times 13.33 = 8864$
$I_{Bm}\ 31.8 \times 13.33 = 424$
F.P. $100 \times 13.33 = 1333$
10621
D.L. Girder $4.75 \times 800 = 3800$
14421

$R3 = 36300$
$M_c = 36300(10.75) - 7300(9.5) - 14500(4.75) = 252000$
$M_b = 36300(6) - 7300(4.75) = 183125$
$M_a = 36300(1.25) = 45375$

Summary of Stresses
FLOOR BMS.

	a	b	c	R3
D.L. Shears	36300	29000	14500	36300
L.L. "	34430	27430	13715	34430
I "	10329	8229	4115	10329
Total	81059	64659	32330	81059
Section Reqd.	8.106□ Gr.	6.466□ Gr.	3.233□ Gr.	8.106□ Gr.
Web used	$32 \times \frac{3}{8} = 12^{\square}$	$32 \times \frac{3}{8} = 12^{\square}$	$32 \times \frac{3}{8} = 12^{\square}$	$32 \times \frac{3}{8} = 12^{\square}$
Moments ft.#				
D.L. Mom.	45375	183125	252000	0
L.L. "	43038	173330	248477	0
I "	12911	51999	74543	0
Total	101324	408454	575020	0
Eff. Depth	2.38'	2.38'	2.38'	
Flange Stress	42600	172000	242000	

	Gr.	Net	Gr.	Net	Gr.	Net	Gr.	Net
Flange Reqd	3.66	2.66	11.76	10.76	16.12	15.12		
Sect. Used								
1/8 Web	1.50	1.50	1.50	1.50	1.50	1.50		
2Ls 6×6×3/4"	16.88	13.88	16.88	13.88	16.88	13.88		
	22.04	18.04	30.14	26.14	34.50	30.50		

End Stiffeners $\frac{81059}{12000} = 6.76^{\square}$" 2Ls 4"×4"×9/16"

Intermediate Stiffeners 3½×3½×3/8 Ls Spaced @ 2'4½"
Used fillers under all stiffeners
Use 7/8" rivets
$\frac{81059}{12000} = 6.75$ Use 10 rivets in end stiffeners
Use 7 rivets in intermediate stiffeners
See Detail drawing AD-1 for more complete details

DESIGN OF HINGE

R for Dead Load = 106600
R " Live " = 42700
R " Impact = 12800
                      162110

Design of hinge Ctd.

Assume pin Dia. = 7"

Bearing area = $\frac{162110}{20000} = 8^{\square}$"

$8/7 = 1\frac{1}{7}$" say $1\frac{1}{2}$"

$\left.\begin{array}{l} \text{Web} = \frac{3}{8}" \\ 2 \text{ Pls} @ \frac{5}{8}" = 1\frac{1}{4}" \\ \hline \text{Total} = 1\frac{5}{8}" \end{array}\right\}$ on Cantilever End.

2 Pin Pls @ $\frac{3}{4}$" = $1\frac{1}{2}$" on suspended End.

End Stiffeners $\frac{162110}{12000} = 13\frac{1}{2}^{\square}$" Use 2Ls $5 \times 5 \times \frac{3}{4}$"

See detail drawing AD-1

$n = \frac{162110}{7880} = 20.6$ Say 21 rivets

$\frac{33}{21}$ = 2 lines of rivets @ 3" o.c.

DESIGN OF END ROLLERS

Abutment end.
Bascule pier end similar

D.L. react	=	109400
L.L. "	=	56500
Impact "	=	16950
		182850

Allowable pressure on Concrete = 400#/□"
Bearing Pl reqd. on abutment = $\frac{182850}{400}$ = 456□"
Use Bearing Pl $21\frac{1}{2} \times 21\frac{1}{2} \times 2$"
Space rollers 1" apart
Bearing on rollers 600D = 600×3 = 1800# per lin. inch
Starting rollers at $\frac{3}{4}$" from ea. end
$21\frac{1}{2}" - 1\frac{1}{2}"$ = 20", 6 rollers Spaced @ 4" c-c.
Cast Steel Pedestal
length of rollers 19"; 6×19 = 114"; 114×1800 = 205000#

PIN

Pin length = 16"; assuming dia. of 4"

$s = \frac{182850}{4 \times 16} = 2857$ #/□" OK

allowable stress 20000 #/□"
Compression area of pedestal not less than
$\frac{182850}{16000} = 11.42$ □" Detail on Drwg. AD-1

R3
D.L. Reaction = 322600#
L.L. " = 56500#
I " = 96780#
474880

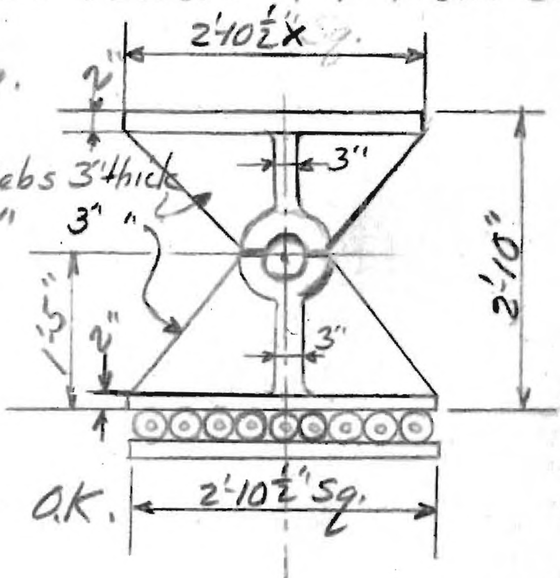
Bearing ℞ req'd on abutment $\frac{474880}{400} = 1188$ □"
" " " = $34\frac{1}{2} \times 34\frac{1}{2} \times 2$"

3" Rollers 600×3 = 1800 #/per inch of bearing
$\frac{474800}{1800 \cdot 30} = 8+$ Use 9 rollers 3" Dia.
Space rollers $\frac{1}{2}$" apart
$9 \times 3 + 8(.5) = 31$" Start rollers $1\frac{3}{4}$" from ends

Pin assume 6" dia.

Compression area of pedestal not less than
$\frac{474800}{16000} = 29.68$ □"

3 Webs 3" thick
4 " 3" "



Req'd. area for pin
$s = \frac{474800}{6 \times 24} = 3292$ #/□" O.K.

Design of web splices
Suspended girder
2 splices will be req'd.

20 feet from R1 & 20 feet from hinge

Design of web splices Ctd.
20' from R1

Mom. D.L.	= ft.# 1,513,500	Shears	D.L.	61400
" L.L.	= 421500	"	L.L.	9400
" I	= 126450	"	I.	2820
" Total	2066450 ft.#	"	Total	73620#

Actual net flange area = $38.11^{\square}$

Mom. taken by web = $\frac{3.23}{38.11} \times 2{,}066{,}450 = 175500$ ft#

Stress in splice Pls = $\frac{175500 \times 12}{42} = 50000\#$

The max. allowed unit stress on the extreme fibre of the girder = 16000 #/□"

Max. allowable unit stress on the splice plates
$= \frac{21}{32} \times 16000 = 10500$ #/□"

$\frac{50000}{10500} = 4.76^{\square}$"   Reqd. 2 Pls 6"×¾"

net area = 9" − 2×2×1×¾" = $6.0^{\square}$"

Allowable rivet stress
$= \frac{21}{32} \times 12000 = 7800\#$

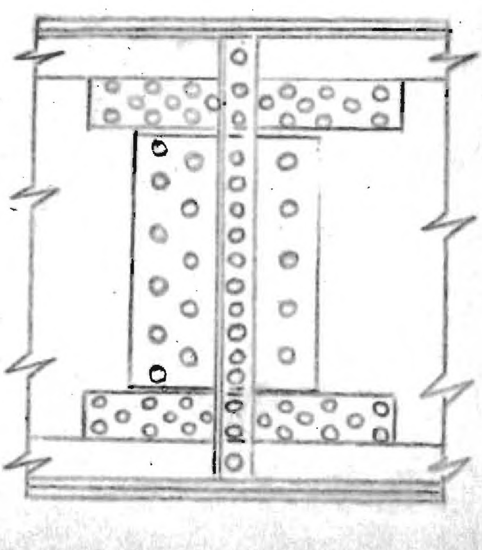
Number of rivets on each side of splice
$= \frac{50000}{7800} = 6.5$ (not less)

Number of rivets reqd. in vertical splice
$\frac{73620}{7800} = 9.45$ on ea. side (not less)

Shear Pl
$\frac{73620}{10000} = 7.3^{\square}$"

Thickness $= \frac{7.3}{2.36} = .11$"
3/4" Pl used. to act also as filler

Rivets $= \frac{73620}{12000} = 6.13$



Length of cover plates

From Johnson Bryan & Turneaure Modern Framed Structures Part III

$$x_n = l\left(\frac{a_1+a_2+\dots a_n}{A}\right)^{\frac{1}{2}}$$

Length of outer cover Pl, $x_1 = l\left(\frac{a}{A}\right)^{\frac{1}{2}} = 60\left(\frac{11}{45.03}\right)^{\frac{1}{2}} = 29.64'$

Adding 8" to ea. end for 2 extra lines of rivets the adopted length becomes 31'-0"

Length of 2nd. cover Pl

$$x_2 = l\left\{\frac{(a_1+a_2)}{A}\right\}^{\frac{1}{2}} = 60\left\{\frac{11+11}{45.03}\right\}^{\frac{1}{2}} = 41.70'$$

Adding approx. 8" to ea. end for 2 extra lines of rivets the adopted Length becomes 43'-0"

Stiffeners at abutments & bascule piers

Total reaction 148530 of this it will be assumed that, due to the deflection of the girder, 60% or 0.60 × 148530 = 89118

Assuming permissible bearing stress at 24000 #/□"

reqd. bearing area of 2 ∠s on the flange ∠s = $\frac{89{,}118}{24000} = 3.71$□" assuming ∠ with 5" net bearing length = $4\frac{3}{8}$"

Reqd. thickness of each stiffener = $\frac{3.71}{2 \times 4.38} = .423$" or $\frac{7}{16}$" with 3" net to web the ∠s will be $5 \times 3 \times \frac{7}{16}$

Use same size ∠s for outer stiffeners. Use fillers

Radius of gyration of 2∠s separated by $1\frac{3}{8}$" about an axis in the plane of the web =

$$r = \left\{\frac{2(2.19(3.44)^2 + 1.12(1.128)^2)}{6.62}\right\}^{\frac{1}{2}} = 2.88$$

Eff. length of stiffener = 40"

allowable stress = $p = 19000 - 100 \times \frac{40}{2.88} = 17600$ #/□"

$\frac{89118}{17600} = 5.08$□" Reqd. area furnished = 6.62 O.K.

Stiffener rivets

Since loose fillers are used, the number of rivets will be increased 1% for each $\frac{1}{16}$" of thickness of fillers or 24%

Safe resistance of one $\frac{7}{8}$" rivet in bearing on $\frac{3}{8}$" web =

$\frac{7}{8} \times 0.375 \times 24000 = 7880$ #

Number of rivets

$\frac{89118}{7880} = 11.33$ not less & exclusive of those in flg. Ls.

Intermediate Stiffeners

Space as shown on drwg. AD-1

Outstanding legs to be about $\frac{1}{30}$ depth of girder $+ 2''$ $\frac{48}{30} + 2 =$ Use $3\frac{1}{2} \times 3 \times \frac{3}{8}$ Ls. with fillers

Flange Riveting

Spacing is computed at center of each panel. Since the angle of slope for the lower chord is small, it will not be taken into account

For the lower chord $p = \frac{vd}{KV}$

For the upper chord $p = \frac{v}{\left\{\left(\frac{KV}{d}\right)^2 + w^2\right\}^{\frac{1}{2}}}$

v = least value or safe resistance of rivet.

V = Total shear at cross section.

w - Vertical load per inch of length applied directly to flange

d = Effective depth of girder

K = A flange stress apportionment factor, representing the ratio of the total stress borne by a part or all, of the flange material proper to the total stress borne by the full flange area, including the web equivalent

Pitch at Support

Least value of v of a $\frac{7}{8}$" rivet on $\frac{3}{8}$" web =

$\frac{7}{8} \times \frac{3}{8} \times 24000 = 7880\#$

Eff. Depth approx. 60"

K for the tension flange = Area of 2-6×6×$\frac{3}{4}$ Ls less 4 1" holes = 13.88 □" ÷ 13.88 + $\frac{1}{8}$ web = $\frac{13.88}{17.11} = .81$

K for Comp. Flange (based on gross area) = $\frac{16.88}{20.11} = 0.84$

$V = 140000\#$

$w = \frac{7000}{12} = 584\#/\text{lin.inch}$ $p = \frac{7880 \times 60}{140000 \times 0.81} = 4.18''$ Spacing 4"

For Top flange

$$p = \frac{7880}{\sqrt{\left(\frac{.84 \times 140000}{60}\right)^2 + 584^2}} = 3.60''$$

Spacing $3\frac{1}{2}$"

Flange riveting Ctd.

At 20 feet from end.

$V = 106600$

K for bott. flange $\frac{35.88}{38.31} = 0.94$

K " top " $\frac{42.88}{45.31} = 0.95$

p for bott. fl. $= \frac{7880 \times 60}{106{,}600 \times .94} = 4.7''$ Use $4\frac{1}{2}''$ Spacing

p for top " =

$$\frac{7880}{\sqrt{\left(\frac{.95 \times 106600}{60}\right)^2 + 584^2}} = 6\tfrac{1}{2}'' \quad \text{Use } 6'' \text{ Spacing}$$

At center of span use 6" spacing for both top & bott. chords.

Cover Plate Riveting

Rivets in pairs opposite to each other. $p = \frac{2vd}{KV}$

$v = 7220$ for $\frac{7}{8}$ rivet

$d = 57''$

K for Bott. flange $= \frac{11}{28.11} = .392$

K for Top flange $= \frac{13}{33.11} = .392$

$V = 140000\#$

p for bott. flange plate $= \frac{2 \times 7220 \times 57}{.392 \times 140000} = 15''$.

Use 6" spacing throughout except for 18 inches at each end where a spacing of $3\frac{1}{2}''$ will be adopted.

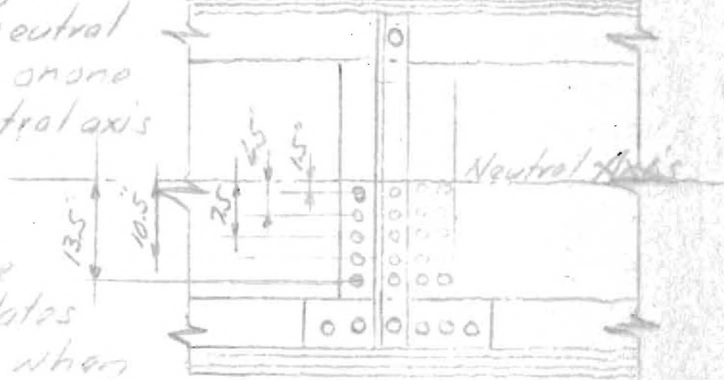
For further details see drawing AD-1

Intermediate Girder

Splices will be made at points f, h & j. at h shear will be zero, since it is the center of the span, therefore, splice will be designed to resist bending only

Moment of resistance of web $= \frac{1}{8} A_w p_c d$
$= \frac{1}{8} \times 41 \times \frac{3}{8} \times 16000 \times 41 = 1{,}265{,}000$ in.#
Use 2 vertical splice plates 12" × ¾" and 2 horiz. plates 5¼" wide on the flange angles.

Since safe stress on flange rivets @ 20.5" from neutral axis = 7880, safe stress on one rivet at 1 inch from neutral axis
$= \frac{7880}{20.5} = 385$



Moment of resistance in vertical splice plates on one side of splice when "y" = vertical distance of any rivet from neutral axis
$= 385[4\{\overline{1.5}^2 + \overline{4.5}^2 + \overline{7.5}^2 + \overline{10.5}^2 + \overline{13.5}^2\}]$
$= 385[4(2.25 + 20.3 + 56.2 + 110.2 + 182)] = 650000$
Difference in moment of resistance to be developed by horizontal splice plates $= 1{,}265{,}000 - 650000 = 615000$

$2n \times 7880 \times 41 = 615000$
$n = \frac{615000}{2 \times 7880 \times 41} = .955$

Use 4 rivets each side of splice spaced at 3" c.c.

The flange rivets must transfer the horizontal increment of flange stress from web to flanges in addition to developing the moment of resistance of web splice.
The average theoretical rivet spacing over splice in top flange taking K = 0.95 =

$$p = \frac{7880}{\sqrt{\left(\frac{.95 \times 0}{41}\right)^2 + \overline{584}^2}} = 13.4$$

Number of rivets req'd. to transmit Hor. increment of flange stress $= \frac{7.5}{13.4} = .56$

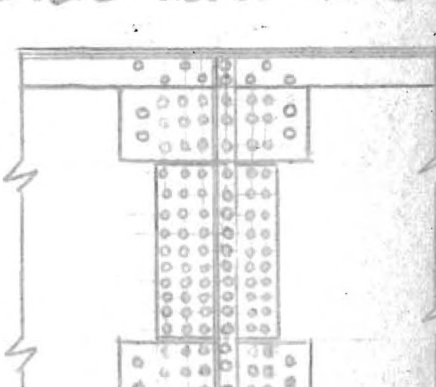
∴ .955 + .56 = 1.515 rivets since never less than 3 rivets should ever be used 3 will be adopted for each side of splice

The horizontal splice plates must transmit the stress carried by .95 rivets = .95×7880 = 7480
Net area of 2 Pls. = 7480 ÷ 16000 = .468□" or .468 ÷ 2 = .234□" for each plate. Use $5\frac{1}{4}\times\frac{1}{4}$" Pls.
5.25×.25 = 1.31□" each, which is satisfactory.

<u>Splices at f & j</u> will be made the same as each other
Moment. Since the D.L. mom. is the greater (see Table of stresses on Sheet A6), the splice will be designed using the full D.L. Mom. = 1,291,000 ft#
Shear = 225820#
Actual net effective flange area = 27.97□"
Bending mom. carried by web = $\frac{3.09}{27.97}$·1,291,000 = 143,000
Assume hor. splice Pls. $12\times\frac{3}{4}$"
Stress in hor. " " = $\frac{143000\times12}{42}$ = 40800#
Max. allowable stress on extreme fibre of girder = 16000#
∴ " " " " Hor. Splice Pls =
$\frac{21}{35}\times 16000 = 9600$ #/□"

Req'd. area splice plates = $\frac{40800}{9600}$ = 4.26□"
or 2 Pls. 12×¼ But ¾" Pls have been adopted because of thickness of flange ∠s. & to stiffen the splice.
Since the fibre stress was reduced on splice plates so the rivet stress will be reduced in proportion to their distance from the neutral axis. $\frac{21}{35}\times7880 = 4720$
Number of rivets on each side of splice = $\frac{40800}{4720}$ = 8.65
" " " " " " " vertical splice Pls.
req'd. to resist shear = $\frac{225820}{7880}$ = 28.6 Use 30 each side

<u>Flange Riveting</u>
Bott. chord $p=\frac{vd}{KV}$



Top chord $p=\frac{v}{\left\{\left(\frac{KV}{d}\right)^2+\omega^2\right\}^{\frac{1}{2}}}$

pitch at support
v = 7880 for ⅞" rivet on ⅜" web
d = 108"
K for the tension flange = Area of 2-6×6×¾" ∠s

less 4 - $\frac{15}{16}$" holes = 13.88□"
1 PL 13×1" less 2 holes - 11.00□"
24.88□"

$K = \frac{24.88}{24.88+5.35} = .823$ for tension flange

K for comp. flange $= \frac{29.88}{29.88+5.35} = .84$
$V = 244450$
$\omega = 584$ #/inch

Tension flange $p = \frac{7880\times108}{.823\times244450} = 4.2"$

Comp. flange $p = \frac{7880}{\sqrt{\left[\left(\frac{.84\times244450}{108}\right)^2 + \overline{584}^2\right]}} = 3.96"$

Spacing at Pier Bott. flange 4" o.c. top flange $3\frac{1}{2}$" o.c.

Point f.

$K = \frac{24.88}{24.88+3.09} = .894$ for tension flange

$K = \frac{29.88}{29.88+3.09} = .910$ for comp. flange

$p = \frac{7880\times63}{.894\times225820} = 2.46$ for tension flange

$p = \frac{7880}{\sqrt{\left[\left(\frac{.91\times225820}{63}\right)^2 + \overline{584}^2\right]}} = 2.38$ for comp. flange

Space rivets in 2 rows Stagger at $2\frac{1}{4}$" o.c. both top & bott. flange

Point f & j

K = same as at Supt. for both tension & comp. flanges
V = 175400
$\omega = 584$ #/inch

Tension Flange $p = \frac{7880\times63}{.823\times225820} = 2.68"$

Comp. Flange $p = \frac{7880}{\sqrt{\left(\frac{.84\times225820}{63}\right)^2 + \overline{584}^2}} = 2.62"$

Space @ 3" o.c. in both tension & comp. flanges. in two rows staggered

Point "g" & e

$K = \frac{24.88}{24.88+2.25} = .918$ for Tension flange

$K = \frac{29.88}{29.88+2.25} = .930$ " comp. "

$$p = \frac{7880 \times 45}{.918 \times 175400} = 2.19'' \text{ for tension flange}$$

$$p = \frac{7880}{\sqrt{\left(\frac{.930 \times 175400}{45}\right)^2 + 584^2}} = 2.49'' \text{ for comp. flange}$$

Space rivets @ 4" o.c. staggered 2 rows to center.

Riveting of cover plates

$$p = \frac{2vd}{KV}$$

At point "e"

$d = 63''$

$v = 7880$ for $\frac{7}{8}\phi$ rivet

$$K = \frac{11}{11 + 13.88 + 3.09} = .394 \text{ for tension flange}$$

$$K = \frac{13}{13 + 16.88 + 3.09} = .395 \text{ for comp. flange}$$

$V = 153400$

$$p = \frac{2 \times 7880 \times 63}{.394 \times 153400} = 16.5''$$ Actual pitch in line of stress must not exceed 6" Therefore pitch in both top & bott. flanges will be 6"

At Support

$d = 100$ $\quad V = 244450$

$$K = \frac{11}{11 + 13.88 + 5.35} = .363 \text{ for tension flange}$$

$$K = \frac{13}{13 + 16.88 + 5.35} = .368 \text{ for comp. flange}$$

$$p = \frac{2 \times 7880 \times 100}{.363 \times 244450} = 17.70''$$

A spacing of 6" will be adopted for both top and bott. flanges for full length of girder

Stiffeners

At supports.

Reaction = 429450# of this (due to the deflection of girder) 60% will be assumed to be carried by each pair of Ls.

$429450 \times .6 = 257670$

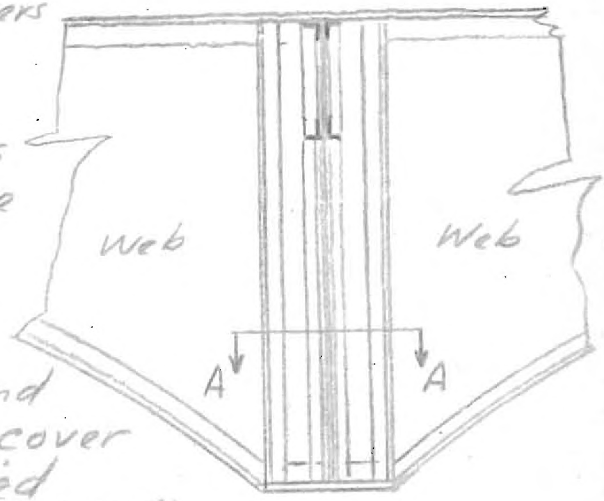
Assuming permissable bearing stress = 24000 #/□"

the reqd. bearing area of 2Ls = $\frac{257670}{24000} = 10.7$ □"

2 Ls $3\frac{1}{2}'' \times 5'' \times \frac{3}{4}''$ with $3\frac{1}{2}''$ leg next to web

Radius of gyration about an axis in the plane

of the web = r = 3.00
Effective length = 50"
The permissable compressive stress = $19000 - 100\frac{50}{3}$
= 17335 #/□" limiting the stress to 13000 #/□"
Reqd. area of 2∠s =
$\frac{257670}{13000} = 19.7$□"
Therefore instead of a single pair of ∠s it will be necessary to provide two pairs as shown in sketch. Also use solid 3/4" filler plate under entire group of stiffeners and floor Beam.



Intermediate Stiffeners
Make same as those used in suspended girder.

Note - in both top and bott. chords the 1st. cover plate will be carried full length of girder both flngs.
2nd. cover plate will be extended from points "e" to "e" over both piers in top flange & from "f" to "j" in bott. flange

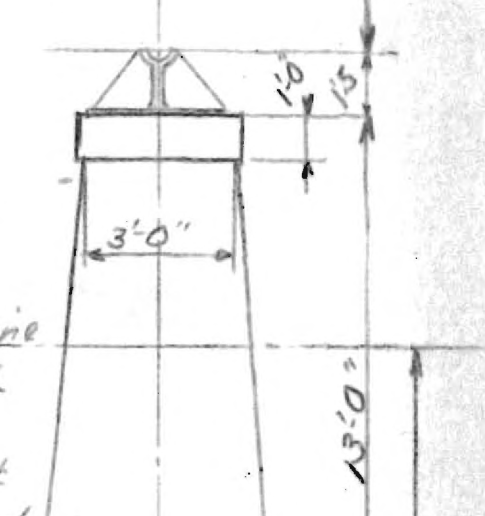
SECT. A-A

Note - Due to the rigidity of the reinforced concrete deck, and also to the gusset plates extending from bottom chords of floor beams to bottom chords of main girders it was assumed that any further bracing to resist wind or sway would be superfluous.
When encasing main girders unusual care must be exercised to keep any gunite from entering hinged joint. This may be avoided by inserting asphaltic board between the two face angles of the joint, removing same after conc. has set.

Design of Piers

Fin Rdway

Weight of conc. 150#/cu. ft.
Reaction at each Pedestal = 429450# Total
" " " D.L. = 356000#
Surface of approach exposed to wind = 8 square ft per lin ft.
Velocity of current at surface = 5 miles per hour.
Ice & drift collected about the upstream end of the pier for a depth of 6' below high water level & for an average width of 18 feet.
Wind pressure 30#/□'



Overturning moments
Wind pressure
$80 \times 8 \times 30 = 19200\#$
Acting at a distance of 22.25' above base of pier.
Overturning moment =
$19200 \times 22.25 \times 12 = 5,126,400$ in#
Pres. on pier $= 3 \times 5 \times 30 = 450\#$
Mom. $= 450 \times 13.5 \times 12 = 7290$ in#
Overturning pres. due to current
$2.96 \times 5^2 = 74\#/□'$ at surface
at bott $= \frac{74}{2} = 37\#/□'$ at bott of drift
$6 \times 18 \times \left(\frac{74+37}{2}\right) = 5994\#$

If p represents the intensity of pressure at the surface, & p_1 the intensity of pressure at the bottom of the ice, and d_1 the distance from the surface to the bottom of the ice, the distance x_0 from the surface to the center of pressure =

Length of pier at cap = 30'-0"
at base = 32'-0"

$x_0 = \frac{(2p_1+p)d_1}{3(p_1+p)}$, $p = 74$, $p_1 = 37$, $d_1 = 6'$

$x_0 = \frac{[2(37)+74]6}{3(37+74)} = \frac{888}{333} = 2.66'$ & 8.34' to bott.

Overturning mom. $= 5994 \times 8.34 = 50000$ ft.#
$= 600,000$ in#

Pressure of water on pier below ice $= 5 \times 5.5 \times 19 = 523\#$
Mom $= 15696$ in.#

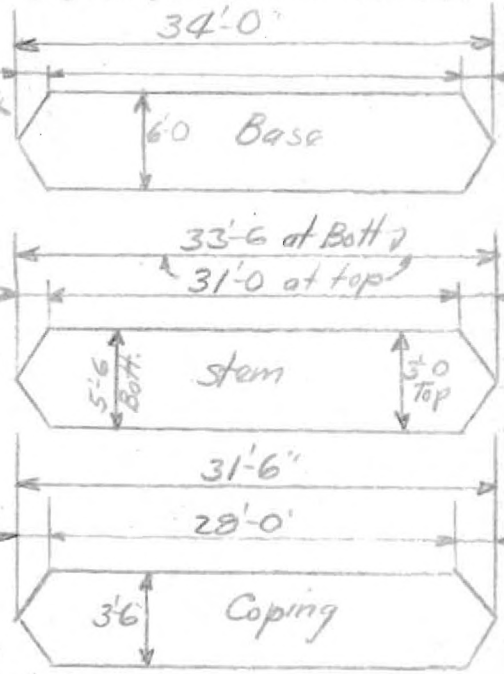
Design of piers Ctd. Sheet #A22

Summary of overturning moments

Wind Pressure on bridge	5,126400 in#
" " " pier	7290 "
Due to current on ice & drift	600000 "
Due to water pressure below ice	15696 "
Total overturning Moment	5,749,386 "

<u>Resisting Moment</u>

Taking moments about the downstream lower edge of ftg.



<u>Wt. of ftg.</u> = 31×6×150 = 27900#

Mom. = 27900×17 = 474300 ft#

<u>Stem.</u>

Wt. = 248550

Mom. = 248550×17 = 4,225,350 ft#

Coping Wt. = 15750

Mom. 15750×17 = 267750 ft.#

It is obvious that, even without taking into consideration the effect of the reaction from the bridge, the pier is safe from overturning, therefore no further calculations will be necessary. (Reinforce both faces with ½# bars 12" o.c. bothways) Pier will rest on rock

<u>Abutments</u>

Reactions from Bridge	D.L. 109400
	L.L. 30100
	I. 9030
See sheet #A23 for sketch	Total 148,530

The back wall from bridge seat to finished Roadway will be designed as a cantilever retaining wall

Hgt. of wall = 11'6"

Width at top 10"

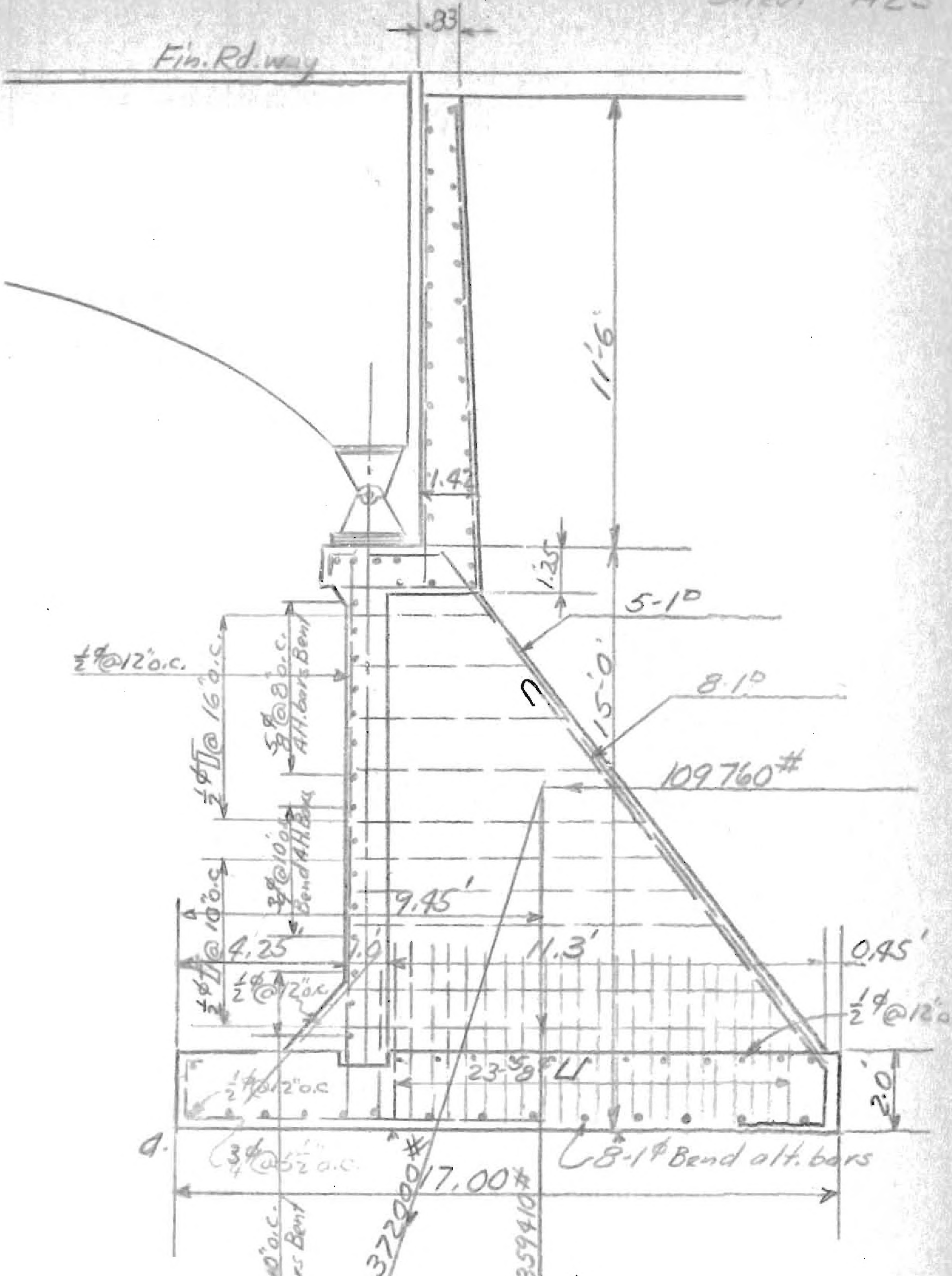
" " base 1'-4"

<u>L.L.</u>

Load on each rear wheel = 10000# this will be assumed as acting over an area of 10×3

L.L per □' = $\frac{10000}{30}$ = 333# + 30% impact = 432#/□'

f_s = 18000, f_c = 700



Note- In main vertical wall, horizontal bars will be placed alt. straight & bent, bent bars to be bent over counter forts & carried to ¼ point of adjacent span
Longitudinal bars in rear portion of ftg. to be handled similarly

Scale of Forces 1" = 100000#
Scale of Dist. ¼" = 1'-0"

Equating this Live Load to an equivalent surcharge taking earth at 100#/cu. ft. $\frac{432}{100}$ = 4.32' equivalent filling

$p = 0.2948wh$ at bott.
$= 0.2948 \times 100 \times 16.33 = 482$ #/ft.
at top
$p = .2948 \times 100 \times 4.32 = 127$ #/ft

$H = .1474 \cdot 100 \cdot \overline{16.33}^2 = 3950$ #

$M = 0.5896wh^3$
$= 0.5896 \cdot 100 \cdot \overline{16.33}^3$
$= 257000$ in #

$bd^2 = \frac{M}{K}$

$d = \sqrt{\frac{257000}{12 \cdot 113}} = 13.72"$

Allowing 3" for steel protection thickness of wall at bott. = 17"

Mom. at 1/3 hgt. from bott.
$= M = 0.5896 \cdot 100 \cdot \overline{12}^3$
$= 101,800$ in #

$A_s = \frac{101800}{18000 \cdot \frac{7}{8} \cdot 12} = .538^{\square"}$

$= \frac{7}{8}φ$ @ 13" o.c.

$\frac{1}{2}φ$ @ 12" o.c. both ways

A_s at bott. $\frac{257000}{18000 \cdot \frac{7}{8} \cdot 14} = 2.2^{\square"}$

$\frac{7}{8}φ$ @ $6\frac{1}{2}$" Stop one half of bars at $\frac{1}{3}$ of wall hgt.

Mom. at $\frac{2}{3}$ hgt. from bott. $= 0.5896 \cdot 100 \cdot \overline{8.12}^3 = 31400$ in. #
$A_s = \frac{31400}{18000 \cdot \frac{7}{8} \cdot 9} = .221^{\square"}$

REAR FACE OF BACK WALL

Keyway $\frac{432}{8 \times 12 \times \frac{7}{8}}$ 51 #

ABUTMENT BELOW BRIDGE SEAT

Hght. from bott. of ftg. to bridge seat = 15 feet.
Total hgt. to top of back wall = 27 feet. See sketch on Sheet A23

The abutment will be designed as a counterfort type reinforced concrete abutment.

Counterforts Spaced at 8'0" o.c.

The base plate will be assumed to be 24" thick

Pressure sustained by lower foot of slab - $P = \frac{1}{3}Wh$

$P = \frac{1}{3} \times 100 \times 31 = 1033$ #/□'

$1033 \times 8 = 8264$# Total load per strip one ft. wide between counterforts

$M = \frac{8264 \times 8 \times 12}{8} = 99168$ in #

$bd^2 = \frac{M}{K}$ $d = \sqrt{\frac{99168}{12 \times 113}} = 8.55$

Allowing 3" for protection of reinforcing $8.55 + 3 = 11.55$

The wall will be made 12" thick throughout.

As at bott. $= \frac{99168}{18000 \times \frac{7}{8} \times 9} = .70$□" Use $\frac{7}{8}\phi$ @ 10" o.c. Bend alternate bars. Use this steel & spacing for 3 ft. up from bott.

$P = \frac{1}{3} \times 100 \times 28 = 934$#, $W = 934 \times 8 = 7472$

$M = \frac{7472 \times 8 \times 12}{8} = 89600$ in #

$As = \frac{89600}{18000 \times \frac{7}{8} \times 9} = .634$□" $\frac{7}{8}\phi$ @ 11" o.c.

$P = \frac{1}{3} \times 100 \times 24 = 800$#, $W = 800 \times 8 = 6400$#

$M = \frac{6400 \times 8 \times 12}{8} = 76800$ in #

$As = \frac{76800}{18000 \times \frac{7}{8} \times 9} = .54$□" $\frac{3}{4}\phi$ @ 10" o.c

$P = \frac{1}{3} \times 100 \times 20 = 666$# $W = 666 \times 8 = 5328$#

$M = \frac{5328 \times 8 \times 12}{8} = 63800$ in #

$As = \frac{63800}{18000 \times \frac{7}{8} \times 9} = .45$□", $\frac{5}{8}\phi$ @ 8" o.c. to top. Bend alt. bars

For temperature bars, $\frac{1}{2}\phi$ @ 12" o.c. will be used at right ∠s to main reinforcing.

$H = 0.1474wh^2 = 0.1474 \times 100 \times 31^2 = 14150$

$14150 \times 8 = 113{,}200$# Total pressure on counterfort

This will act at $\frac{1}{3} \times 26.5 = 8.82$ ft. from bottom.

Mom. in counterfort $= 113200 \times 8.82 \times 12 = 12{,}000{,}000$ in #

The counterforts will be made 16" wide

Depth at top of ftg. = 11.3 feet

" " Bridge seat = 2.5 "

The counterfort can be designed as a T beam since the vertical slab forms a T with the counterfort.

$A_s = \dfrac{12000000}{18000 \times \frac{7}{8} \times 131} = 7.64^{\square}$ 8-1$^{\square}$ Bars will be used

Moment at bridge seat

$= 0.5896 \times 100 \times 15.81^3 = 233000 \times 8 = 1,865,000$

$A_s = \dfrac{1,865,000}{18000 \times \frac{7}{8} \times 25} = 4.74^{\square''}$ 5-1$^{\square}$ bars

8-1$^{\square}$ bars will be carried for 2/3 of the height of counter fort 3 bars will be hooked at this point & the remaining 5 bars will be continued up into the bridge seat & hooked.

Test for stability against overturning (See Sheet A23)

Pressure against wall

$= .286 \times \dfrac{100 \times 31^2}{2} = 13720\#$

Reactions from bridge

D.L. $= \dfrac{109400 \times 2}{26} = 8420\#/ft.$ Per Ctf. $= 8420 \times 8 = 67360\#$

L.L $= \dfrac{30100 \times 2}{26} = 2320\#/ft.$ " " $= 2320 \times 8 = 18560\#$

I $= 2320 \times .4 = 925\#/ft.$ " " $= 925 \times 8 = 7400\#$

L.L. + I $= 3245\#/ft.$ " " $= 3245 \times 8 = 25960\#$

L.L. + D.L. + I $= 11665\#/ft.$ " " $11665 \times 8 = 93320\#$

Dist of pt. of application of the above loads from toe $= 4.75$ feet

Footing $= 8 \times 2 \times 17 \times 150 = 40800\#$ c of g. from a $= 8.5'$

Main wall $= 13 \times 8 \times 1 \times 150 = 15600\#$ " " " $= 4.75'$

Back " $= 8 \times .83 \times 12.75 \times 150 = 12700\#$ " " " $= 6.75'$

" " $= \dfrac{8 \times .59 \times 12.75 \times 150}{2} = 450\#$ " " " $= 7.36'$

Counterfort $2.3 \times 11.75 \times 1.33 \times 150 = 5400\#$ c of g " " $= 6.41'$

" $\dfrac{.9 \times 11.75 \times 1.33 \times 150}{2} = 10500$ " " " $= 10.75'$

Earth prism $2.3 \times 11.75 \times 100 \times 8 = 21600$ c.g 6.41 from a

" " $8 \times 9.42 \times 24.5 \times 100 = 185000$ " 12.29 " a

Sum of vertical loads including D.L from bridge but not bridge L.L. & Impact L.L is included on pavement up to abutment approaching bridge

$= 67360 + 40800 + 15600 + 12700 + 450 + 5400 + 10500 + 21600 + 185000 = 359410\#$

Taking moments about toe or pt. a

Location of resultant of vertical loads

$(67360\times 4.75+40800\times 8.5+15600\times 4.75+12700\times 6.75+450\times 7.36+5400\times 6.41+10500\times 10.75+21600\times 6.41+185000\times 12.29)\div 359410 = 3{,}386{,}000\div 359410 = 9.45'$

The resultant falls within the middle third therefore the abutment is safe against overturning.

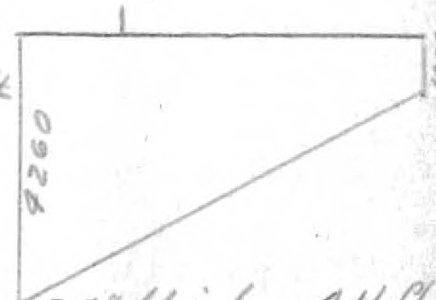
Pressure at toe $=(4\times 17-6\times 6.75)\dfrac{359410}{17^2} = 27.5\left(\dfrac{359410}{17^2}\right) = 34200$

$\dfrac{34200}{8} = 4260$ #/ft. of toe

Pressure at heel $(6\times 6.75-2\times 17)\dfrac{359410}{17^2} = 6.5\left(\dfrac{359410}{17^2}\right) = 8060$, $8060\div 8 = 1000$ #/ft. of heel.

Design of toe

Av. pres. $=\dfrac{4260+3400}{2} = 3830\#$



Mom. $=\dfrac{3830\times 4.25^2\times 12}{2} = 415000$ in#

$d=\sqrt{\dfrac{415000}{12\times 113}} = 17.5''$

The Ftg. was assumed to be 24" thick. All ftgs. should have a min. of 4" protection for reinforcing steel $\therefore 17.5+4 = 21.5$ say 22"

$v=\dfrac{4.25\times 3830}{12\cdot \frac{7}{8}\cdot 18} = 86.2$ which is rather high for

Concrete without special provision for diagonal tension

Using a depth of ftg. of 24" total depth

$v=\dfrac{4.25\times 3830}{12\times \frac{7}{8}\times 20} = 77.6$ a 45° fillet will be provided as shown on sketch Sheet #A23 this makes the shear at edge of fillet =

$v=\dfrac{3.25\times 3830}{12\times \frac{7}{8}\times 20} = 59\#/□''$ and by inspection it can be seen that the shear will be less at face of wall.

$A_s = \dfrac{415000}{18000\times \frac{7}{8}\times 20} = 1.31□''$ $\frac{3}{4}\phi$ @ $5\frac{1}{2}''$ o.c.

longitudinal shrinkage reinforcement $\frac{1}{2}\phi$ @ 12" o.c.

Design of rear portion of ftg.

Since the weight of the rear portion will not act unless the wall is about to overturn & since the

the calculations show that the abutment is stable against overturning, only the weight of the earth with its superimposed Live Load plus impact will be considered in designing the rear portion of ftg.

$2.3 \times 11.75 \times 100 = 2700$
$9.42 \times 29.00 \times 100 = 27300$
$30{,}000\#$

The ftg. will be a beam continuous over the counterforts.

$$M = \frac{30000 \cdot \overline{8}^2 \cdot 12}{12} = 1{,}920{,}000 \text{ in}\#$$

$$d = \sqrt{\frac{1{,}920{,}000}{11.75 \times 12 \times 113}} = 11''$$

$$u = \frac{30000 \times 4}{141 \cdot \frac{7}{8} \cdot 7} = 86\# \text{ which is excessive without stirrups}$$

For the sake of rigidity the ftg. will be given an effective depth of 20", u then becomes

$$\frac{30000 \times 4}{141 \times \frac{7}{8} \times 20} = 48.9\#/\square'' \text{ Stirrups will be unnecessary}$$

$$A_s = \frac{1{,}920{,}000}{18000 \times \frac{7}{8} \times 20} = 6.08\square'' \quad 8\text{-}1\square, 4\text{ Bt.} \& 4\text{ St.}$$

There will be a tendency for the wall slab to tear away from the counterforts, this stress will vary with the depth & will be pure tension since the counterforts must be provided with reinforcing to resist shrinkage stresses, stirrups will be used extending from back of counterforts thru & into the wall.

Tension at bott. of wall $= 8264\#$, $\frac{8264}{18000} = .458\square''$
2 legs of $\frac{1}{2}\square$ stirrups $= .38\square''$
Place $\frac{1}{2}\square$ $\sqcup$ @ 10" o.c. for $\frac{1}{3}$ of hgt. of counterfort
At $\frac{1}{3}$ hgt. Tension $= 5328\#$ $\frac{5328}{18000} = .296\square''$
Place $\frac{1}{2}\square$ $\sqcup$ @ 16" o.c. to bridge seat
There will also be a force tending to tear the counterfort away from the rear portion of footing

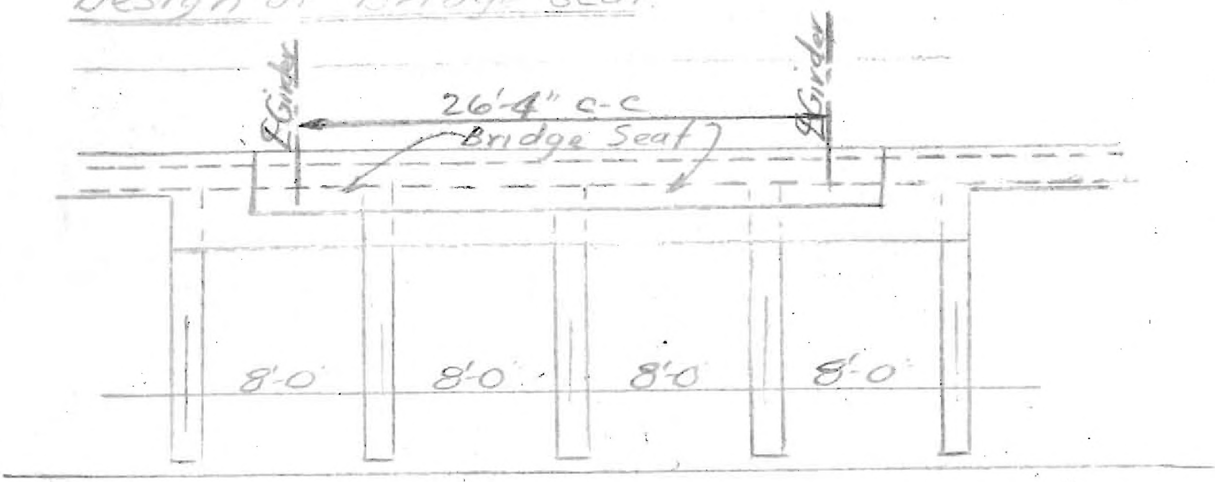
Design of Abutments Ctd Sheet # A29

Weight of earth = 30000 × 8 = 240000 #

$\frac{240000}{18000} = 13.3^{\square''}$ 2-$\frac{5}{8}\phi$ legs = .60$^{\square}$

$\frac{13.3}{.60} = 22$ Use 22 $\frac{5}{8}\phi$ ⊔ embed 2'0" into counterfort

Design of Bridge seat



PLAN OF ABUTMENT

Total D.L.+L.L.+I = 141820 # Concentrated on wall
This load will be distributed by means of the bridge seat. To accomplish this 4-$\frac{3}{4}\phi$ × 8'0" bars will be placed under bridge seats

Area of pedestal = 2.835 × 2.835 = 8.04$^{\square}$

$\frac{141820}{8.04} = 17650$ #/□

Taking an effective area of 9×12 = 108 □", then $\frac{17650}{108} = 163$ #/□"
Which is a very low unit comp. stress

The wings are designed in the same manner as the abutment.
Foundation is assumed to be rock & will be scored to provide resistance to sliding

Back filling against abutment is not to be placed until after all dead load of bridge is in place